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SOME HYDRAULIC CHARACTERISTICS OF COARSE-BED RIVERS

BY

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies for acceptance a thesis entitled SOME HYDRAULIC CHARACTERISTICS OF COARSE-BED RIVERS submitted by VICTOR J. GALAY in partial fulfilment of the requirements for the degree of Doctor of Philosophy.

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ABSTRACT

Data from the field and laboratory, dealing with the behaviour of coarse-bed rivers, are analyzed under the following topics: the sampling and analysis of coarse bed-material, the threshold of motion condition, the amount of material in transport, the resistance to flow, the design of stable channels, and the depth of scour at river bends.

The adequate sampling of a coarse river-bed should consist of a subsurface scoop sample, borings and a surface grid sample. Analysis of a wide range of field data indicated that the largest stones in a mixture start to move at lower velocities than the same size stones in a uniform bed of rip rap. The analysis of flume experiments using dimensional analysis resulted in the conclusion that the relative depth is a significant parameter in assessing the rate of bed-load transport. A comparison of the measured bed-load transport in the Elbow and North Saskatchewan Rivers to computed transport indicated that the existing formulas are inadequate. The measurement of the actual protrusion height of stones on a river-bed resulted in the development of a flow resistance equation, for rigid beds, which is somewhat dissimilar from the formulas of Keulegan and Kellerhals. Formulas relating stable channel width,

depth, and slope to discharge and size of bed-material were developed for coarse-bed channels. The sounding of river bends before, during, and after floods revealed that scour holes at bends were relatively stable; filling of the scour hole during recession of the high flows did not take place. Analysis of a number of forced and free bends led to the development of tentative design formulas for assessing the maximum depth of scour. This maximum depth of scour was related to the width, radius and internal angle of the river bend.

The importance of more adequate field data was stressed and recommendations regarding field and laboratory investigations were formulated.

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CHAPTER I

INTRODUCTION

Engineers today are being confronted with increasingly complex problems in river engineering. In particular, the training of rivers in mountainous areas has called for increased attention because of the construction of vast hydro-electric and diversion schemes, pipelines, and recreation facilities. Unfortunately, most of the river training experience that engineers have accumulated is related to sand-bed rivers and is inadequate for dealing with coarse-bed mountainous rivers. The term coarse-bed river refers to a river having an immobile or a mobile-bed of non-cohesive material larger than 2 mm. in size. This bed-material is generally classified into gravel (2 mm. to 2.5 inches),cobbles (2.5 inches to 10 inches), and boulders (over 10 inches).

Although the behaviour of coarse-bed rivers was discussed in the nineteenth century by DuBoys (1879), few field investigations have been conducted to date. An investigation by Lane and Carlson (1953, 1954) yielded useful information on the design of canals in this material. Through a cooperative effort by the University of Alberta,

the Alberta Department of Highways, the Water Resources Division and the Alberta Research Council, investigations were carried out to assess the character of rivers in Alberta and British Columbia (Qureshi 1962, Kellerhals 1963, and Van Der Giessen 1966). The study by Kellerhals extended the knowledge in regard to stable channel design and resulted in new design equations. Subsequent investigations by Hollingshead (1968a, 1968b) yielded information on bed-load discharge. Investigations on sixteen rivers in Pennsylvania by Brush (1961) resulted in information on the relation of the river channel to the geologic characteristics of the basin. Fahnestock (1963) studied unstable coarse-bed streams flowing from glaciers and obtained information on the shifting of channels and the stability of the bed-material. Investigations on rivers in Europe, in particular by Meyer-Peter (1949) and by Ramette and Heuzel (1962), were concerned primarily with the applicability of the Meyer-Peter and Muller bed-load formula.

As part of the continuing river research program in Alberta the North Saskatchewan River at Drayton Valley was investigated during the summers of 1965 and 1966 (Galay 1967a). Information regarding the shifting of the main channel and the movement of large bed-material during near-bankfull flows was obtained. Subsequent longitudinal

soundings on the North Saskatchewan River downstream of Edmonton, the Oldman River and the Athabasca River resulted in valuable data concerning scour at river bends (Hollingshead and Schultz, 1968). The location of the investigations is shown in FIGURE 1. A limited field program was carried out on Wilson Creek in Manitoba in 1968 to assess the flow conditions necessary to move coarse bed-material.

The Alberta information combined with data from other investigations will be analyzed in depth to yield formulas that may be of assistance to engineers. Specific problems receiving attention are:

(1) What bed sampling techniques are best for immobile or mobile-bed rivers?

(2) Given the size distribution of the bed-material, under what flow conditions will it begin to move?

(3) What is the best method for estimating the amount of bed-material in transport under known flow conditions?

(4) What is the resistance to flow for an immobile or a mobile-bed of coarse material?

(5) Given the expected flow conditions and the size distribution of the bed-material, what width,

depth and slope will result in a stable channel?

(6) What is the maximum depth of scour that can be expected at a river bend?

It is emphasized that the lack of data prevents the answers to these problems from being conclusive and compels the exclusion of the consideration of several important problems, for example, the design of training works. After the analysis of existing data, specific recommendations are presented to guide future investigations.

CHAPTER II

SAMPLING AND ANALYSIS OF BED-MATERIAL

2.1 INTRODUCTION

The purpose of sampling and subsequently analyzing the bed-material in coarse-bed rivers is three-fold:

(1) To assess the size of bed-material that will just begin to move under known flow conditions, (threshold of motion), as well as ultimate degradation;

(2) To estimate the volume of material transported along the bed under known flow conditions;

(3) To assess the roughness or the resistance due to friction that
 (a) an immobile bed offers to the flow,
 (b) a mobile bed offers to the flow.

In all the above cases it is necessary to know a representative size as well as the variation in size encountered on the river-bed. This chapter will discuss the basic types of bed-material samples that can be obtained, the various sampling techniques with their appropriate methods of analysis, and the relation between the purpose of sampling and the available sampling techniques.

2.2 TYPES OF BED-MATERIAL SAMPLES

A review of the literature on coarse-bed rivers reveals that there are many techniques used in sampling the river-bed, and that the various sampling techniques

yield three basic types of samples:

(1) Volumetric (or bulk)

(2) Areal (or surface)

(3) Combination of volumetric and areal.

The basic types of samples along with the associated sampling techniques are summarized in TABLE 1. These techniques will now be discussed in more detail.

2.3 VOLUMETRIC SAMPLING TECHNIQUES

2.3(a) Scoop Sampling Technique

This technique is rather straightforward with an appropriate sampler being used to scoop up material from the bed of a river (Inter-Agency Committee on Sedimentation 1963). Material can also be shovelled from open bars into containers. This results in a sample having a mixture of surface as well as sub-surface material; this technique can, however, be refined so that only surface or sub-surface material is picked up. PHOTOGRAPH 1 shows a sub-surface scoop sample being obtained at 6 inches below the surface of an open bar.

2.3(b) Square-Depth Sampling Technique

In this case all the material to a depth equal to the largest stone within a small enclosed area on an open bar is collected. PHOTOGRAPH 2 shows a three foot square framework prior to the removal of the enclosed material.

TABLE I

SUMMARY OF BED-MATERIAL SAMPLING TECHNIQUES AND ANALYSIS

Type of Bed-Material Sample	Sampling Technique	Measurements Taken	Analysis
Volumetric	(1) Scoop (2) Square-depth	Weights by sieve	Plot % retained by weight vs. sieve size
Areal	(1) Grid (2) Line (3) Square-surface	Intermediate or all three axes of each stone or weight of stones in certain size classes	Plot % finer than by number vs. intermediate axis which is equivalent to % of bed covered by material finer than a given size vs. intermediate axis or % finer than by weight vs. intermediate axis
Combination	(1) Transect	Intermediate axis of all stones larger than 2 inches, weights by sieve of finer material	Plot % of bed covered by material finer than given size vs. diameter (see Wolman 1955)

This technique also results in a sample having a mixture of surface and subsurface material (Van Der Giessen 1966, p. 37).

2.4 AREAL SAMPLING TECHNIQUES

2.4(a) Grid Sampling Technique

This sampling technique was initially devised by Wolman (1954) and quoting directly consists of:

> In the desired reach of the stream a grid system is established either by pacing or with actual lines. The size of the grid is determined by the length of reach which the sampler desires to describe. ... After establishing the grid, ... 100 individual pebbles are picked up from the bed (at designated grid points). The sampling is probably less subjective when lines or tapes rather than pacing are used to fix the individual sampling points. Randomness in the selection of each pebble can only be obtained if the sampler tries not to look at the bed as he picks up each pebble. The author's practice is to draw each pebble from beneath the tip of the toe of his boot.

This sampling technique has been widely used in North America, in some cases with slight modifications. Brush (1961) noted that 60 pebbles were adequate for a sample. Various other investigators using this technique were Qureshi (1962) on the Red Deer River, Kellerhals (1963) on rivers in British Columbia, Fahnestock (1963) on the White River and Van Der Giessen (1966) on streams in Western Alberta. PHOTOGRAPH 3 illustrates the pacing technique in grid sampling.

Several investigators (Kellerhals 1963) have noted that the sample obtained by pacing is slightly different from that obtained by using a grid with established lines. This discrepancy is due to two factors:

(1) The operator's toe usually falls on a stone projecting from the bed instead of in the space between the stones resulting in a larger stone being picked up;

(2) In picking up a stone beneath an operator's toe the finger usually comes in contact with the larger stones before the smaller ones, even though these larger stones may not be directly below the toe.

2.4(b) Line Sampling Technique

This technique is essentially a minor modification of the grid sampling technique in that pebbles are picked up at regular intervals along an outstretched tape instead of at grid points. The sample should be identical to that obtained by the grid sampling technique and is well suited for sampling along river banks or on top of long narrow bars (PHOTOGRAPH 4).

2.4(c) Square-Surface Sampling Technique

A square-surface sample is obtained by picking up all the surface pebbles within a small enclosed area on an open bar. This area is taken to be representative of the whole channel bed. This technique was initially

used by Lane and Carlson (1953) in their studies of the San Luis Valley Canals.

A variation of this technique, which was used on the North Saskatchewan River (Galay 1967a), is to photograph the stones in an enclosed area with some scale in the photograph (PHOTOGRAPH 5). The stones usually orient themselves so that the long axis is perpendicular to the flow and the intermediate axis parallel. These two axes can be measured with sufficient accuracy directly from the photograph. This photographic technique has also been used by Pashinskiy (1964), Church (1968), and Ritter and Helley (1969).

2.5 COMBINATION SAMPLING TECHNIQUES

The sampling techniques in this section could be combinations of the volumetric and areal techniques previously discussed. There are several possible combinations but only that developed by Wolman (1955) will be discussed. This combination technique consists of obtaining four scoop samples of material finer than 2 inches along a transect (line across a channel) with all the pebbles larger than 2 inches being measured directly.

2.6 ANALYSIS OF BED-MATERIAL SAMPLES

A sample of river bed-material obtained by one or several of the previously mentioned techniques has to be analyzed and described in a reasonable numerical manner.

Engineers are generally interested in a representative size of the sample as well as its dispersion or sorting.

2.6(a) Volumetric Samples

A scoop sample, a square-depth sample and the finer portion of Wolman's (1955) combination sample are normally sieved and weighed in order to arrive at a distribution of sizes. The results from a sieve analysis are generally plotted on a graph with cumulative % finer (ordinate) versus the sieve size (abscissa) as shown in FIGURE 2. In some cases it is more convenient to plot the sieve results on logarithmic-probability paper as shown in FIGURE 3. Natural sands often plot as straight lines on this type of paper (Blench 1952, Vanoni et al 1960), but this is not true for all coarse materials as they may be bimodal. The log-probability plot facilitates the determination of percentile values.

2.6(b) Areal Samples

As previously mentioned a grid sample should be identical to a line sample and for the forthcoming discussion only a grid sample will be mentioned. Also, a grid sample obtained by pacing will usually yield pebbles having a larger size than a sample obtained from established grid points (FIGURE 4). Therefore, wherever possible, a grid sample should be obtained from an established grid instead of by pacing.

It should also be noted that a grid sample analyzed by number will yield an entirely different curve than an analysis of the same sample by weight (FIGURE 5). The analysis by weight is greatly influenced by any large cobbles in the sample. There does not appear to be any good reason why a grid sample, which is areal and therefore two-dimensional, should be analyzed by weight which is based on volume and therefore three-dimensional. Although the grid sample has been analyzed by weight by several investigators it is recommended that this analysis procedure be discarded for grid samples.

The square-surface sample can be analyzed by number (taking into account all the stones within the enclosed area), by weight and by percent of the bed area (from a photograph) covered by a certain size or smaller The resulting distribution curves may be quite different from each other. This may be better understood by considering a hypothetical square-surface sample of 100 spheres of various sizes as shown in TABLE 2. The sizes to be considered range from 0.5 inches to 5.0 inches and there are 10 spheres of each size in the sample. This square-surface sample is analyzed in the three different ways mentioned previously and plotted in FIGURE 6. The plotted points are joined by smooth lines instead of

TABLE 2

ANALYSIS OF SQUARE-SURFACE SAMPLE SPHERES

Number	Size (inches)	Cum. % Finer than by Number	Projected Surface Area (Sq. in.) $10\chi\pi/4D^2$	Cum. % of Area Covered by Size Finer than	Volume (Cu. in.) $10\chi\pi/6D^3$	Cum. % Finer than by Volume (or weight)
10	0.5	10.0	2.0	0.3	0.6	0.0
10	1.0	20.0	7.9	1.2	5.2	0.3
10	1.5	30.0	17.6	3.7	17.7	1.2
10	2.0	40.0	31.4	7.8	41.9	3.3
10	2.5	50.0	49.0	14.3	81.6	7.4
10	3.0	60.0	70.6	23.6	142.0	14.6
10	3.5	70.0	96.2	36.4	225.0	26.0
10	4.0	80.0	125.3	53.0	335.0	42.9
10	4.5	90.0	159.0	74.0	476.0	67.0
10	5.0	100.0	196.0	100.0	655.0	100.0
Σ=100			755.0		1980.0	

broken lines in order to appear similar to conventional distribution curves. The analysis by number yields representative diameters smaller than would be obtained from the weight or percent of bed area analysis. The latter two analysis techniques yield distribution curves that are similar.

In grid sampling, which is most common, the probability of picking up a certain stone is dependent upon its surface area and it should be possible to demonstrate that a grid sample analyzed by number is similar to a square-surface sample analyzed by percent of bed area covered. FIGURE 7 illustrates that a grid sample analyzed by number does yield a similar curve to a photographic square-surface sample analyzed by percent of bed area. It is, therefore, not surprising that Kellerhals (1967) found that a square-surface sample analyzed by weight yields very similar distribution curves to a grid sample analyzed by number. Ritter and Helley (1969), using a size particle-size analyzer, came to a similar conclusion. A paced grid sample, from the Chowchilla River, analyzed by number agreed closely with a square-surface sample (from photograph) analyzed by volume (FIGURE 8). The volumetric analysis is equivalent to the weight analysis if all the particles have identical specific weights. The volume of a particle, which has its intermediate axis determined from a photograph, can be calculated by multiplying the cube of the intermediate axis by

a particle shape coefficient (Ritter and Helley, 1969, p.6). It would therefore be expected that a photographic sample (square-surface) analyzed by volume should agree closely to a sieve analysis of the same surface particles. Ritter and Helley (1969), however, state:

> For each of the three samples (two of coarse sand, one of ash slough) the median diameters determined by sieve analysis are smaller-almost half in two samples - than those determined by the particle-size analyzer.

This statement may be in error as the particles that were analyzed may not have been identical; the sieve analysis may have been carried out on a "depth" sample while the volume analysis with the particle-size analyzer would be on a surface sample. These two samples could be distinctly different from each other. Ritter and Helley (1969) do not clearly describe how the sieve samples were obtained.

Also, the grid sample analyzed by number appears to yield the most useful distribution curve as it conveys a picture of river's bed. One can visualize the meaning of the following statement: "80% of the river's bed is covered by pebbles equal to or smaller than 2 inches".

2.6(c) Combination Samples

Wolman's (1955) combination sample, as previously discussed does not give a definite picture of the surface or sub-surface of a river-bed and will not be dealt with further.

2.7 IMPORTANT BED-MATERIAL PARAMETERS

In studying the behaviour of the river-bed engineers are generally interested in a representative size as well as the dispersion of the material that makes up the bed.

2.7(a) Representative Size

There are several measures used in arriving at a representative size of bed-material, the most common being the median or D_{50} size. The median size is the 50% size on a cumulative frequency curve and defines the size separating the sample into two equal halves. The median size is the easiest to determine but since it is not influenced by the distribution of particle sizes in a sample its use as a measure of representative size is not recommended (Folk 1965, 1966).

Another common measure is the arithmetic mean which is obtained by adding up all the material sizes and dividing by the number of values. This measure is, however, difficult to derive for material finer than 1/2 inch and a number of investigators resort to the arithmetic mean by weight which is equal to $\Sigma \frac{PiDi}{100}$ where Pi = % in class interval of mean diameter

Di. This latter measure is also referred to as the arithmetic mean computed by the method of moments. The determination of the arithmetic mean, by any technique, is, however, time consuming.

Folk (1965) recommends the use of a graphic mean diameter which is based on geologic phi units. In using percentile values directly from log-probability graphs this measure would become the geometric mean and could be determined by:

$$D_G = \sqrt[3]{D_{84} \times D_{50} \times D_{16}} \qquad (2.1)$$

Folk (1965) states that this measure corresponds very closely to the mean as computed from the method of moments, yet it is much easier to use. However, the study of distribution curves for coarse materials has shown that the geometric mean is equal to the median for samples which are log-normal (Galay 1967a). This can be shown by considering the distribution curve for sample A in FIGURE 9. Using equation 2.1 the geometric mean is:

$$D_G = \sqrt[3]{2.69 \times 1.0 \times 0.37} = 1.0 \text{ inches}$$

which is identical to the median diameter. The arithmetic mean is, however, equal to 1.55 inches. A second distribution curve for sample B is shown in FIGURE 9 to illustrate that both the median and the geometric mean, being equal to 1.0 inches for both samples, are poor measures of representative

size. However, the geometric mean does approximate the arithmetic mean fairly closely for bimodal distributions (Folk 1965).

A convenient measure of representative particle size would be (Galay 1967a):

$$D_m = \frac{10\% + 30\% + 50\% + 70\% + 90\%}{5} \text{ sizes} \quad (2.2)$$

The representative size as obtained from the above relation is reasonably close to the arithmetic mean as shown by comparing the values for the two samples in FIGURE 9:

Sample	Arithmetic Mean (in.)	Dm (in.)	D_{50} (in.)	D_G (in.)
A	1.55	1.45	1.00	1.00
B	1.03	1.02	1.00	1.00

More investigation into representative particle size is necessary in order to make final recommendations. Presently, however, most of the data as reported by various investigators lists the median as the representative particle size; it is this value that will be used in subsequent analysis.

The mode, which is the most frequently occurring particle size, is rarely used by engineers although it is useful in assessing the source of moving sediment in natural rivers.

2.7(b) Dispersion

Trask's sorting coefficient = $\sqrt{D_{75}/D_{25}}$, using millimeter sizes, is the most common engineering measure of sorting or dispersion but it measures only the sorting in the central part of the distribution curve. A better measure as noted by Simons and Richardson (1966) would be the standard deviation or gradation which is given by the formula:

$$\sigma_b = \tfrac{1}{2}\left[\frac{D50}{D16} + \frac{D84}{D50}\right] \qquad (2.3)$$

This measure would be especially applicable to log-normal distributions and would still be a reasonable measure of dispersion for distributions that are not log-normal.

2.8 LOCATION OF SAMPLES

The knowledge of the variation in sizes of bed-material with location on the river-bed is of importance. The pool and riffle sequence in coarse-bed rivers has been well documented (Leopold, Wolman and Miller 1964) with the bed material in riffles being larger than that of pools. It would be advantageous to obtain samples from open bars, from pools and from riffles and average the results from these three samples in order to arrive at a representative picture of the river-bed. Sampling the river-bed in pools, especially during high flows, may, however, be somewhat hazardous.

It is interesting to note that the distribution of material in transport as collected by a bed-load sampler resembles the distribution of a sample obtained by the square-depth sampling technique (Hollingshead 1968a). A comparison of the two distribution curves is shown in FIGURE 10. However, more samples of various types from a variety of coarse-bed rivers would be necessary in order to make conclusive statements.

2.9 SAMPLING AND ANALYSIS TECHNIQUES IN RELATION TO THE PURPOSE OF SAMPLING

As outlined in the introduction the purpose of sampling the material on the bed of a river is three-fold and each purpose will now be discussed in more detail.

2.9(a) Threshold of Motion

In order to assess what particles will begin to move along the river-bed under known flow conditions it is essential to have a distribution curve of the particles making up the surface layer. In this instance the sample could be obtained by a grid technique and analyzed by number. Sampling along river or channel banks is also of importance in assessing bank stability. A grid sample analyzed by number should suffice here.

Ultimate degradation of a river is achieved when the small bed particles have been removed leaving a layer of particles that are too large to move. The remaining particles would be very close to the threshold of motion state.

The sampling of the river-bed, however, to assess degradation, should take place at various depths below the bed surface; core samples analyzed by weight would be necessary here.

2.9(b) Sediment Transport

Once the surface layer of particles is in motion the sub-surface particles will be carried off in a rush and the moving material could range from fine sand to boulders. A distribution curve of sub-surface material would be necessary in order to assess the volume of material in motion when using a sediment transport formula. This material could be obtained by a scoop sample at some depth below the surface and would be analyzed by weight. A square-depth sample (from an open bar) analyzed by weight should also yield a representative picture of the moving bed material.

2.9(c) Resistance to Flow

It is generally known that the resistance to flow of an alluvial river bed depends on whether a bed is immobile or mobile. Each of these aspects will now be discussed.

2.9(c)i Immobile River-bed

In this case only the surface material is relevant and a grid sampling technique with subsequent analysis by number could be used. It is important here that a clear picture of the make-up of the bed surface be given in order that a representative size could be utilized in a flow formula.

2.9(c)ii Mobile River-bed

The resistance to flow may depend on the amount of material moving along the bed if bed forms are generated and may vary from the case with no moving particles. Sub-surface material would be moving along with surface material and sampling should be by scoop with analysis by weight.

2.10 CONCLUSIONS

After reviewing the sampling and analysis techniques in relation to the purpose of sampling it becomes apparent that river-beds should be sampled in the following two ways, unless it is certain that the bed is immobile:

(1) Scoop or core sampling at some depth below the surface, and

(2) Grid sample obtained by a taped grid or by pacing.

The two samples so obtained should be adequate for assessment of threshold of motion, sediment transport and resistance to flow.

The scoop sample, which is volumetric, should be analyzed by weight while the grid sample, which is areal, should be analyzed by number. A grid sample analyzed by number conveys a clear picture of the surface material on a river bed as the probability of picking up a certain stone is dependent upon its surface area.

After reviewing the various measures of representative size it appears that the arithmetic mean is the most reasonable one but is time consuming to use. Equation 2.2 has been used by the author and found to be convenient. The standard deviation, equation 2.3, should be used as a measure of dispersion. Some of the measures noted by sedimentary petrographers are not especially useful to engineers.

2.11 RECOMMENDATIONS

Studies should be conducted to arrive at the most convenient measure for representative size of bed-material. The relationship between the median size by weight to the median size by number should be investigated further.

The variation in size and dispersion of bed-material with the type of morphological feature in the river channel could certainly receive more attention. The various types of morphological features such as bars, pools and riffles are spatially distinct indicating distinct processes which may result in distinct types of coarse material deposits.

CHAPTER III

FIELD OBSERVATIONS ON THE THRESHOLD OF MOTION FOR COARSE BED-MATERIAL

3.1 INTRODUCTION

Recently there have been many papers dealing with the theoretical and experimental aspects of the threshold of motion of particles located on the bottom of a river or a channel (ASCE Task Committee on Sedimentation Manual 1966, Neill 1967, Coleman 1967, Egiazaroff 1967, Neill and Yalin 1969). These papers have presented detailed analysis of the phenomenon as investigated in laboratory flumes. An excellent history of this topic has been presented by Leliavsky (1955).

The engineer is interested in the threshold of motion of coarse bed-material for several reasons which can be grouped into two categories:

Category A - Coarse Mixture on Bed

(1) In designing stable channels the channel must be designed so that neither its bed nor its banks experience excessive erosion.

(2) The application of several bed-load transport formulas in rivers requires the knowledge of a critical shear stress.

Category B - Uniform Bed-material

(1) In designing river training works and hydraulic structures the use of riprap is common. The riprap must be so designed that the surface layer of stones will not move under design conditions.

The design of riprap protection (Category B) has been discussed by many engineers (Ramette 1963, Blench 1969, Peterka 1963, California Dept. of Public Works 1960, Izbash and Khaldre 1970) and will not be dealt with here. However, the various criteria developed for the threshold of motion for coarse mixtures will be compared with criteria that pertain only to uniform particles.

In this chapter an attempt will be made to arrive at criteria for movement of coarse mixtures which are based on flow information that is generally available to river engineers such as mean flow velocity in a cross-section, surface width, mean-flow depths, bed material composition and water surface slope. Field data obtained by the writer from the North Saskatchewan River and Wilson Creek, as well as other investigators' data will be analyzed.

3.2 THRESHOLD OF MOTION FIELD DATA

During the summers of 1965 and 1966 measurements of the largest stones moved on the bed of the North Saskatchewan River at Drayton Valley were undertaken with the

hope of correlating this information to the existing flow parameters (Galay 1967a). Subsequent measurements in 1968 were taken on Wilson Creek in Manitoba with the same purpose in mind. This information will now be summarized along with similar information obtained by other investigators.

3.2(a) North Saskatchewan River near Drayton Valley

APPENDIX 2 presents pertinent geomorphic and hydraulic information concerning the North Saskatchewan River near Drayton Valley. Measurements of the largest stones moved by the 1965 peak-flow were carried out after the flow receded. These stones were located on open bars adjacent to the low flow channel and their sizes and locations are tabulated in TABLE 3. All the stones were considered to be part of the river-stone population and not "erratics". The distribution of surface material is shown in FIGURES 4 and 7; the largest stones moved cover approximately 0.01% of the bar surface. It would appear that this 1965 flood was capable of moving virtually all the material making up its bed.

Subsequent to the stone survey, cobbles having intermediate diameters of 2, 3, 4 and 6 inches were painted various colours and placed in separate "squares" on an open bar, (PHOTOGRAPH 6). This procedure was carried out

TABLE 3

SIZES AND WEIGHTS OF STONES
MOVED BY 1965 PEAK FLOW
NORTH SASKATCHEWAN RIVER NEAR DRAYTON VALLEY

Location	Dominant Lithology	SIZE (Intermediate Axis) Average (in.)	SIZE (Intermediate Axis) Maximum (in.)	WEIGHT Approximate Average (lb.)	WEIGHT Approximate Maximum (lb.)
U/S of #82	Quartzite	8.0	10.0	42	75
U/S of #86	Quartzite	9.8	14.0	60	100
U/S of #87	Quartzite	8.4	11.0	-	-
D/S of #88	Quartzite	7.5	10.2	35	55
U/S of #89	Quartzite	8.6	11.5	43	90
#94	Quartzite	7.9	11.0	32	50
#96	Quartzite	11.0	14.0	75	100
#97	Quartzite	9.5	10.0	50	55
D/S of #101	Sandstone	10.5	16.5	80	120

before the 1966 peak flow, with the hope that the water would rise sufficiently to displace some of the cobbles from the bar surface. Velocity measurements directly over the squares were to be carried out with the hope of correlating the maximum measured velocity with the largest cobble moved. However, the peak flow in 1966 was so high that more than just the top layer of cobbles were moved; in fact, the whole bar was eroded and moved downstream. It was apparent that stones larger than six inches were moved making velocity - stone size correlations impossible.

More data on the threshold of motion was made available when a sacked-concrete revetment at x-sec 93 failed. The river flows through the Pembina Oil Field and the purpose of the revetment was to protect a water injection well. PHOTOGRAPH 7 a, b, and c shows the stages of failure of the revetment, while PHOTOGRAPH 8 shows the extent of the channel shift. After the high flow receded many of the concrete sacks were found on top of the first and second point bars downstream from x-sec 93 as shown in FIGURE 11 and PHOTOGRAPH 9. They were moved as far as a mile and were located on both sides of the main flow channel. The concrete sacks were measured and weighed and correlated to a critical mean velocity as obtained by $Vm_c = Q/A$ using the area of x-sec 94 as a reference since a large number of sacks passed

through this section. However, the concrete sacks would be set in motion whenever they were eroded from the revetment which is dissimilar from stones on a river bed. Some concrete sacks were not moved from the location of the revetment; therefore it was assumed that the flow was just sufficient to move the sacks (critical state). The data are tabulated in TABLE A-5, APPENDIX 3.

3.2(b) Wilson Creek, Manitoba

Wilson Creek is an experimental catchment on the escarpment of the Riding Mountains (MacKay and Stanton 1964). In the summer of 1968 a beaver dam was blown up in the upper reaches resulting in a flow of 1,000 cfs (100 yr. frequency) passing down the creek. A number of stations were established to obtain the following data (Newbury, 1968):

(1) The peak flow level and the local water surface slope from stage rods 200 ft. apart.

(2) The local surface velocities at peak flow stage by using wooden surface floats.

(3) The largest stones moved by the peak flow. The largest stones were located accurately in the 200 ft. reach and painted. After the flow subsided the stones were checked for movement.

(4) Cross-sections before and after peak flow in order to check for aggradation or degradation and to obtain mean flow depths.

TABLE 4 - WILSON CREEK

AVERAGE HYDRAULIC AND CROSS-SECTION

PROPERTIES AT PEAK FLOW

STAGE - 1968

STATION	AVERAGE AREA FLOW ft^2 (A)	AVE. TOP WIDTH ft^2 (bw)	MEAN FLOW DEPTH A/bw (ft)	W.S. SLOPE	SURFACE FLOAT VELOCITY (ft/s)	τ (lb/ft^2)
No. 1	89.5	33.6	2.7	0.0350	14.3	5.92
No. 4	94.2	30.8	3.1	0.0345	11.6	6.65
No. 6	62.8	30.0	2.1	0.0055	10.5	0.72

TABLE 5 - WILSON CREEK

DIMENSIONS OF MOVED BOULDERS

THAT ARE SMALLER THAN FLOW DEPTH

STATION	BOULDER DIMENSIONS (ft.) MAJOR	INTERMEDIATE	MINOR AXIS
No. 1	2.3	2.0	1.6
	1.9	1.8	1.3
	2.2	2.0	2.0
No. 4	2.8	2.5	2.0
	2.0	1.8	1.5
	3.0	3.0	2.0
	3.7	3.0	2.6
	2.2	1.9	1.2
	2.5	2.4	2.4
	2.6	2.4	2.2
	3.2	3.0	3.0
	2.1	2.0	1.4
	3.1	2.8	2.2
No. 6	2.2	1.8	1.8
	2.4	1.8	1.6
	1.9	1.8	1.6
	1.9	1.8	1.6
	2.0	1.8	1.4
	1.6	1.4	1.4
	1.0	0.9	0.9
	1.7	1.6	1.4

TABLE 4 presents the average hydraulic and cross-section data for stations 1, 4 and 6, as they were the only locations having stone size data and TABLE 5 gives the sizes of moved stones. The representative boulder size for each station was obtained by averaging all the moved stones that were smaller than the depth of flow. These were then tabulated with the corresponding mean velocities in TABLE A-5 (APPENDIX 3). The mean velocities were obtained by multiplying the surface float velocities by 0.87 (after Fahnestock 1963). Several stones larger than the depth of flow were also moved, but the distance moved was very short suggesting that the motion was by "canting". The bed of Wilson Creek contains material that is predominantly in the cobble range as shown on the bed-material distribution curves (FIGURE 12).

3.2(c) <u>Middle Fork Eel River (Ritter 1967)</u>

The Middle Fork Eel River flows from the western slopes of the Coastal Ranges in Northwestern California. A study was undertaken to determine the maximum size of bed-material that will be moved by different velocities of flows. At intervals of 10 - 20 feet along a cross-section at a gauging station, twelve 1-foot squares of bed-material were painted and located with reference to the right bank. A flow of some 3,750 cfs having an average velocity of 6 ft. per second removed approximately

50% of the stones in the painted squares. The stone sizes and the corresponding velocities are tabulated in TABLE A-5 (APPENDIX 3). The bed-material ranged in size from sand to boulders and averaged 1.0 inches.

3.2(d) Truckee River, California-Nevada (Birkeland, 1968)

Glacial outwash along the Truckee River was found to contain large boulders and gravel bars indicating that exceptionally large flows took place in the valley. An attempt to estimate the mean velocities and tractive forces required to move large boulders was made by computing Manning's roughness coefficient from data collected after the failure of the St. Francis Dam in California. Manning's n was computed to be 0.08 for the St. Francis Dam failure and 0.05 to 0.03 for present day flood flows. Instead of using n = 0.08 a roughness value of 0.06 was thought to be reasonable and this value was used to obtain velocities as tabulated in TABLE A-5 (APPENDIX 3). The data presented here may not be as reliable as some of the previously mentioned cases as no direct measurements of actual flows, water levels, etc., were possible.

3.2(e) White River, Washington (Fahnestock, 1963)

Data on the velocities required to transport coarse materials was compiled by Fahnestock in a study of the processes of valley train formation by a proglacial

stream. Velocities were measured directly by a current meter or with floats depending on the depth of flow. The corresponding sizes of stone in motion were obtained by catching the stones on a screen or by sampling on nearby exposed gravel bars.

3.2(f) <u>Rubicon River, California (Scott and Gravlee, 1968)</u>

On December 23, 1964 a torrential rainfall on the upper part of the Rubicon River caused the failure of the partly completed Hell Hole Dam. The resultant surge produced exceptionally high peak discharges which eroded the valley walls and transported a vast quantity of boulders. Water level and depth measurements were utilized to compute the tractive force, and the mean diameter of the 10 largest stones at the corresponding sites were correlated to this tractive force. The data are tabulated in TABLE A-5 (APPENDIX 3).

3.2(g) <u>Flooding Rivers in Connecticut (Wolman and Eiler 1957)</u>

A flood in August 1955 inundated a large portion of the valley bottoms in Connecticut. Erosion of channel and valley bottom resulted in the movement of cobbles and boulders. Reconnaissance surveys after the flood resulted in a compilation of the sizes of boulders moved in channels having various discharges, slopes, velocities, and depths. The discharges were based on indirect measurements and mean velocities were obtained by dividing

discharge by cross-sectional area. The slope was obtained from topographical maps. There were some twelve different sites investigated along seven different rivers. The data are tabulated in TABLE A-5 (APPENDIX 3).

3.2(h) Coffee Creek, California (Stewart and LaMarche 1967)

On December 22 and 23, 1964 a flood having a magnitude of 17,800 cfs, unprecedented in the 110 year period of settlement, occurred on Coffee Creek destroying large areas of meadowland. Cross-sections surveys were conducted to assess the erosion and deposition features of the flood. From these cross-sections and a knowledge of the peak discharge a mean velocity was computed and correlated to the maximum size stones moved in the vicinity of the cross-section.

3.2(i) Neri River, Japan (Oishi 1956)

In order to assess the transport of gravel and boulders in mountainous streams a number of stones were charged with cobalt-isotope with the intention of finding these stones after recession of the high flows. In 1953, a typhoon struck the area causing a small flood to take place. The movement of the stones was somewhat irregular during the flood. Measurements of the mean velocity, depth and slope were obtained.

3.2(j) Gravel Rivers in Western Alberta (Van Der Giessen 1966)

Four coarse-bed rivers, namely the Castle, Sheep, Elbow and Drywood Creek, were investigated in order to assess their character. Colored river-bed stones were used to study the threshold of motion condition and the distance of transport of bed-material in the Elbow and Drywood Creek. It was, however, difficult to sort out the data presented in this thesis; plots of particle size D_{90} versus tractive force are shown, but the computed tractive force was for a representative flow of a smaller magnitude than the maximum flow that took place in the various rivers. The data were adjusted and are tabulated in TABLE A-5 (APPENDIX 3).

3.2(k) Elbow River, Alberta (Hollingshead 1968a)

Field investigations were carried out on the Elbow River near Bragg Creek in order to learn more about bed-load transport in coarse-bed rivers and to assess the behaviour of this river. Hollingshead (1968a) assessed the threshold of motion state using values of bed shear stress τo_c as determined from velocity profiles. The values used in this study are, however, obtained by using $\tau o_c = \gamma d_* S$ in order to be comparable to the previously mentioned investigations.

3.2(l) San Luis Valley Canals (Lane and Carlson 1953)

A number of stable coarse-bed canals were investigated in order to establish criteria for design of

stable canals. In this case the bed-material was assumed to be at the threshold of motion state for the maximum sustained flows.

3.2(m) Coarse-bed Rivers in British Columbia (Kellerhals 1963)

Seven stable reaches were investigated in order to obtain regime type formulas. Channel dimensions, slopes and bed-material data were collected with formative or bank-full discharges being estimated; the bed-material was assumed to be at the threshold of motion. The data are tabulated in TABLE A-5 (APPENDIX 3).

3.3 ANALYSIS OF FIELD DATA

The field data described in the previous section involving bed-material mixtures will be analyzed in the following ways:

(a) Mean critical velocity related to representative stone size.

(b) Critical shear stress related to representative stone size.

(c) Shields criteria relating a tractive force coefficient to the particle Reynolds number.

(d) Shields tractive force coefficient related to the relative depth of the channel.

(e) Froude number related to the relative depth of the channel.

The relationships obtained from the various treatments will then be compared with corresponding relationships for uniform materials.

3.3(a) Mean Critical Velocity Related to Stone Size

One of the earliest attempts to assess the threshold of motion for particles on the bed of a channel was by Brahms (1753) who presented the equation:

$$Vmc = a \quad Wp^{1/6} \qquad (3.1)$$

where:

Vmc = critical velocity

a = empirical constant

Wp = weight of the particle

This is the simplest approach to the problem but it disregards several significant factors such as the depth of flow. The equation can be modified by assuming a spherical particle which leads to the weight being proportional to the cube of the diameter and:

$$Vmc = a\ D^{1/2}$$

where:

D = particle diameter

A plot of the mean critical velocity versus the corresponding stone size is shown in FIGURE 13. It should be noted that the velocity is the mean velocity for a cross-section, not the mean velocity in a vertical at some designated point in a channel. Also, the stone size is generally the

mean of the largest particles that have been moved; in some cases the stone size has been taken as the D_{90} size, where 90% of the particles are finer than the specified size. The D_{90} particle size was chosen for rivers or canals that had developed a stable non-moving bed.

The numbers adjacent to the plotted points are mean depths of flow.

A tentative equation representing the field data for bed-material mixtures would be:

$$Vmc = 8.0\ D^{1/3} \tag{3.2}$$

The scatter shown on the plot could be attributed to a number of different factors:

(1) Difficulty in obtaining accurate flow data during high flow conditions.

(2) With low values of relative depth the flow would be highly turbulent with a possibility of high concentrations of fine particles. This condition, which may be termed a mudflow, was aptly described by Richardson (1968):

> they saw the moving mass of water and rock coming over the end of the glacier ... described the material as similar to a huge mixture of concrete except that it was darker in color. They stated that the force was so great that immense boulders were thrown from ten to thirty feet into the air as the mass moved forward.

Under these circumstances the values of flow density ρ, specific weight γ, and dynamic viscosity μ would be different from pure water.

(3) The representative particle size was not consistent in all cases as some investigators listed only the one largest stone moved while others took an average of a number of large stones.

It would now be in order to compare equation 3.2 with similar equations for uniform size particles. Bhowmik and Simons (1970) compared the equations obtained by Smith and Hallmark (1965), the USBR (Peterka 1963), and Izbash (1936) as shown in FIGURE 14. All the equations are of the form $V \propto D^{1/2}$ and the following design equation was proposed by Bhowmik and Simons (1970):

$$V_{mc} = 10.4\ D^{1/2} \qquad (3.3)$$

The equations shown in FIGURE 14 are based on uniform particles, however, some of the plotted points pertain to bed-material mixtures.

Equation 3.2 for bed-material mixtures is compared with the various equations for uniform materials in FIGURE 15. The curve for the mixtures is somewhat flatter and intersects the curves for uniform material around the one-foot stone size. It would appear that higher velocities are required

to initiate motion of mixtures in the gravel size range. The converse applies in the boulder range; the D_{50} size will be in motion at lower velocities for mixtures as compared to uniform particles.

Neill (1967) also compared various other equations which could be reduced to a relationship between velocity and stone size if a flow depth is known. The equations compared here are those of Mavis and Laushey (1948), USSR/ Lane (1955), Sundborg (1956), California Highways (1960) and Straub (1953). The velocity used in these equations is, however, not the mean velocity in a cross-section but the mean in a vertical which is assumed to be adjacent to the moving stones. Equation 3.2 for bed-material mixtures is compared to the various equations in FIGURE 16 for material having a specific gravity of 2.65 and a depth of one metre (3.28 feet). An equation by Neill (1968b) is also presented in this plot.

The curve for bed-material mixtures does not agree closely with the curves from other investigators. However, this is not surprising since a mean cross-section velocity was used in arriving at equation 3.2 while a mean velocity in a vertical was used by the investigators noted on FIGURE 16. The slope of equation 3.2 does agree quite well with the slopes of the other curves (except for the California Highways curve).

3.3(b) Critical Shear Stress Related to Stone Size

The critical shear stress has been used as the basis for stable canal design in coarse materials (Lane, 1955). A plot of the critical shear stress versus stone size, shown in FIGURE 17 reveals considerable scatter; no attempt was made to obtain an equation from this plot.

Several relations obtained by other investigators, namely Lane (1955), Kellerhals (1963), Egiazaroff (1965) and Komura (1967) are added to FIGURE 17. It is somewhat difficult to make meaningful comparisons, however, as most of the curves are related to a D_{50} size while the plotted data do not correspond to any one frequency value. It would appear that the relation established by Lane (1955) is the most reasonable. The data indicates that the 1 on 1 slope of the various curves may be too flat; more reliable data would be required to firmly establish the slope.

3.3(c) Shields Tractive Force Coefficient Related to Particle Reynolds Number

The most common criteria for the threshold of motion is based upon the Shields diagram (Shields 1936). Shields plotted data, from flume experiments, on a dimensionless plot having a "tractive-force coefficient" $\tau c/\gamma_s' D$ versus the particle Reynolds number $v_* D/\nu$ where τ_c = the critical shear stress at the bed, γ_s' = the bouyant weight of the bed-material, D = particle diameter, v_* = shear velocity and ν = kinematic viscosity. The "tractive-force

coefficient" has also been termed an "entrainment function" (Randkivi 1967), a "mobility number" (Yalin 1965), and a "dimensionless critical shear stress" (ASCE Task Committee on Sedimention Manual 1966). Since the term "tractive-force coefficient" has not been used extensively by research workers and the mobility of a particle is related to the dimensionless parameters, the term "mobility number" will be used in the remaining discussion.

Shields used physical reasoning to arrive at his dimensionless parameters. However, it is possible to obtain these same parameters by using dimensional analysis and assuming that the threshold of motion is determined by τ_c, γ's, D, ρ and μ where ρ is the fluid density and μ is the dynamic viscosity (ASCE Task Committee on Sedimentation Manual 1966). The choice of variables by the Task Committee is somewhat incomplete; a more complete presentation including all the pertinent variables is presented in section 3.3(e). Shields original diagram is shown in FIGURE 18 with a band through the plotted points. The influence of the particle Reynolds number becomes negligible for $v_*D/_\nu > 70$ (Yalin 1966) and it would appear that the mobility number would have a value ranging from 0.04 to 0.06.

The field data is plotted on FIGURE 19 and shows a wide range of mobility numbers, $\frac{\rho v_*^2}{\gamma' s D}$. The values of the

mobility number are plotted on a histogram, FIGURE 20; the most frequently occurring value (mode) is 0.015 while the mean of all the obtained mobility numbers is 0.042. The use of a mobility number equal to 0.03, also recommended by Neill (1968b) for uniform particles, would appear to be reasonable.

3.3(d) Shields Tractive Force Coefficient Related to the Relative Depth of the Channel

In using dimensional analysis to formulate equations for the threshold of motion it has been argued that the mean depth of flow d_* as well as particle density ρs should be included as important variables (Neill and Van Der Giessen 1966, Yalin 1965). This would result in the following:

$$\tau_c = f_1 (\gamma' s, D, \rho, \mu, \rho s, d_*) \qquad (3.4)$$

The critical shear stress τ_c can be replaced by v_* from $v_* = \sqrt{\tau_c/\rho}$, which leads to:

$$\frac{\rho v_*^2}{\gamma' sD} = f_1 \left(\frac{\rho v_* D}{\mu}, \frac{d_*}{D}, \frac{\rho s}{\rho}\right) \qquad (3.5)$$

Within the range of existing experimental data the density ratio $^{\rho}s/\rho$ appears to be insignificant and for coarse particles the influence of particle Reynolds number may not be influential ($\frac{\rho v_* D}{\mu} > 70$). This results in:

$$\frac{\rho v_*^2}{\gamma s' D} = f_2 \left(\frac{d_*}{D}\right) \qquad (3.6)$$

The form of the mobility number was changed slightly by Neill and Van Der Giessen (1966) by replacing v_* with the

mean critical flow velocity V_{mc}:

$$\frac{\rho V_{mc}^2}{\gamma_s' D} = f_3 \left(\frac{d_*}{D}\right) \tag{3.7}$$

The collected field data were plotted according to the above relationship (FIGURE 21) and shows considerable scatter. Neill's (1968) equation:

$$\frac{\rho V_{mc}^2}{\gamma_s' D} = 2.0 \left(\frac{d_*}{D}\right)^{1/3} \tag{3.8}$$

is also shown on the plot but does not pass through the plotted points. This is to be expected, however, as the definition of mean velocity is not identical - the field data is based on mean cross-section velocities while Neill's (1968) equation is based on mean velocities in the vertical.

3.3(e) Froude Number Related to the Relative Depth of Flow

In using dimensional analysis there are many possible combinations of the variables involved. The following variables are involved in the threshold of motion:

$$\tau_c,\ v_* \text{ or } V_{mc} = f\ (\rho,\mu,D,\rho_s,\sigma_b,fc,fg,b,d_*,Cw,g) \tag{3.9}$$

where the new variables are:

σ_b - shape of bed-material distribution curve

fc = factor defining cross-sectional shape of channel

fg - factor defining plan geometry of channel

b = width of channel

Cw = concentration of suspended particles

Using v_{mc} as the dependent variable conventional dimensional analysis leads to:

$$\frac{V_{mc}}{\sqrt{gd_*}} = f_1 \left(\frac{\rho v_{mc} D}{\mu}, \frac{b}{d_*}, \frac{d_*}{D}, \sigma_b, fc, fg, Cw, \frac{\rho s}{\rho} \right) \qquad (3.10)$$

All the flume and field data have been obtained from channels that were relatively straight, wide ($b>5d_*$) and of a similar cross-section which would remove the b/d_*, fc and fg parameters from equation 3.10.

The effects of the following parameters σ_b, $\frac{\rho s}{\rho}$ and Cw are assumed to be insignificant, primarily because they have not been adequately studied to date and equation 3.10 becomes:

$$\frac{V_{mc}}{\sqrt{gd_*}} = f_2 \left(\frac{\rho V_{mc} D}{\mu}, \frac{d_*}{D} \right) \qquad (3.11)$$

This arrangement is very similar to an arrangement proposed by Blench (1969):

$$\frac{V_{mc}}{\sqrt{gd_*}} = f \left(\frac{^3\sqrt{vgD}}{\nu}, \frac{d_*}{D} \right) \qquad (3.12)$$

for conditions of bed-load charge approaching zero. The term $\frac{\sqrt[3]{vgD}}{\nu}$ is similar to the particle Reynolds number and is called the particle Vig Number. It may be argued that the Reynolds number is insignificant for fully rough turbulent flows which results in equation 3.11 becoming:

$$\frac{V_{mc}}{\sqrt{gd}} = f_3 \left(\frac{d_*}{D} \right) \qquad (3.13)$$

Blench (1970) deletes the g term from the Froude number and arrives at a zero bed factor $F_{bo} = V_{mc}^2/d$; equation (3.13) can, therefore, be modified to:

$$\frac{V_{mc}^2}{d_*} = F_{bo} = f_4 \left(\frac{d_*}{D}\right) \qquad (3.14)$$

The field data for threshold of motion is plotted with F_{bo} versus d_*/D in FIGURE 22 and exhibits the same degree of scatter as the plot with the modified mobility number (FIGURE 21). The equation:

$$F_{bo} = 29 \, (D/d_*)^{1/2} \qquad (3.15)$$

presented by Blench (1967) is plotted on the figure and appears to represent the plotted points reasonably well.

3.4 CONCLUSIONS

After examining the plots of the river data it would appear that the relationship between mean velocity and stone size:

$$V_{mc} = 8.0 \, D^{1/3} \qquad (3.2)$$

can be used to assess when large stones in a mixture will begin to move. The equation is inadequate academically but represents the data as well as the more refined plots which include more variables. A stable natural coarse-bed channel could therefore be designed so that the resultant mean velocity will be less than that indicated by equation 3.2, using D_{50} as the representative diameter.

Equation 3.2, for bed-material mixtures does not agree closely to equations developed for uniform materials

which are of the form $V_{mc} \propto D^{1/2}$. It would appear that higher velocities are required to move a D_{50} size for mixtures as compared to uniform materials in the gravel size range. The converse is the case in the boulder range.

The critical tractive force criterion as developed by Lane (1955) also appears to correspond reasonably well to the field data plotted in this study.

The various threshold criteria that can be developed through dimensional analysis, such as Shields entrainment function or mobility number and the regime theory zero bed factor, were generally reduced to a relationship between a flow intensity parameter and relative depth. The various plots to test these relationships indicated considerable scatter. However, an equation presented by Blench (1967) appears to fit the data reasonably well:

$$F_{bo} = 29\ (D/d_*)^{1/2} \qquad (3.15)$$

3.5 RECOMMENDATIONS

The procurement of more field data on the threshold of motion under well controlled conditions is desirable. The radioactive labelling of various sizes of coarse material in straight canals such as the San Luis Valley canals and the observation of the corresponding state should yield valuable data. Hydraulic parameters would be relatively easy to obtain and the movement of labelled stones could be followed

closely by appropriate detectors.

Laboratory studies to assess the threshold state for bed-material mixtures would also be of value. Almost all laboratory studies conducted to date have dealt with uniform materials. Threshold studies should be conducted with the bed-material naturally sorted by flowing water as is the case in nature. The bed should not be placed and molded by templates or screeds. However, the assessment of the threshold state would probably be very difficult with mixtures as fine material in motion may obscure the observation of the bed-material. It would then be necessary to label certain particles by paint or radioactive tracers and inspect the bed after each test run. These laboratory tests should be carried out for several distinctly different bed mixtures using various density materials. These experiments would, however, be very complex and time consuming.

CHAPTER IV

THE TRANSPORT OF COARSE BED-MATERIAL IN FLUMES AND RIVERS

4.1 INTRODUCTION

To date, most of the laboratory research and field investigations concerned with sediment transport have been confined to relatively fine materials, namely sand (between 0.062 and 2.0 mm in size). Recently, however, the construction of engineering projects adjacent to coarse-bed mountain rivers has resulted in a number of questions that require immediate answers. Some of the more important questions would be:

(1) How quickly will reservoirs fill up if most of the material in transport is coarse?

(2) How much degradation will take place in the river channel below dams?

(3) How much scour will take place near obstacles such as bridge piers, abutments, and river bends?

(4) What is the best design approach for a stable channel that is required to pass large quantities of coarse material?

The answers to some of these questions may exist for sand-bed rivers in the form of empirical equations, but the extrapolation of these equations to rivers having bed-

material as large as four to six inches may be unwise. The answers depend upon an intimate knowledge of the manner and the rate of bed-material transport.

The manner of transport of coarse bed-material may differ somewhat from sand as the fall velocity law is different for the two materials. Another important factor may be the effect of Reynold's number (temperature) which for sand channels greatly influences the amount of material in transport. This may not be the case for coarse-bed channels.

Laboratory experiments dealing with coarse bed-material transport have been carried out by Gilbert (1914), Meyer-Peter and Muller (1948), U.S. Waterways Experiment Station (1935), Liu and Carter (1935) and Bogardi and Yen (1938). These data will be analyzed after examining the transport phenomena from the dimensional analysis approach. A summary of the sediment properties is shown in TABLE 6.

The application of the presently available data to the transport of coarse material in rivers will also be discussed.

4.2 DIMENSIONAL ANALYSIS

The movement of coarse bed-material along the bed of a river or channel is a two-phase phenomenon with the bed-load transport dependent upon the flow conditions, the channel geometry and the properties of the fluid and bed-

TABLE 6

SUMMARY OF SEDIMENT PROPERTIES COARSE BED-LOAD EXPERIMENTS

(After Cooper and Peterson, 1968)

NUMBER	DATA SOURCE	DATE	SERIES NO.	SPECIFIC GRAVITY	MEDIAN DIAMETER (mm) (by weight)	GRADATION	PARTICLE SHAPE
6	USEWS	1935	9	2.65	4.10	1.40	Sub-rounded to sub-angular
10	T.Y. Liu	1937	1	2.66	4.30	1.16	Rounded
			2	2.66	3.25	1.10	Rounded
			3	2.66	2.26	1.13	Rounded
			5	2.66	3.60	1.21	Rounded
22	Gilbert	1914	F	2.69	3.17	-	Sub-rounded
			G	2.69	4.94	-	
			H	2.69	7.01	-	
23	Meyer-Peter and Muller	1948	1	2.68	28.65	-	Well-rounded
			2	2.68	5.21	-	Well-rounded
			3	2.68	4.50	D_{90}=12.3mm	Well-rounded
			4	2.68	3.30	Mixture	Well-rounded
			5	2.68	2.00	Mixture	Well-rounded
24	Bogardi and Yen	1938	1	2.63	10.0	-	Well-rounded
			2	2.61	6.8	-	Well-rounded
			3	2.64	15.0	-	Well-rounded
3	Meyer-Peter and Muller	1948	1	1.25	5.21	Lignite	-
			1	4.22	5.21	Barite	-

material (see FIGURE 23).

The variables can be assembled in the following manner:

Flow conditions:

qs = weight of bed-load transported per unit time per unit width

d_* = mean flow depth equal to flow area divided by water surface width

S = slope of energy gradient

C_w = concentration of suspended particles

Channel geometry:

b = width of channel

fc = factor defining cross-sectional shape of channel

fg = factor defining plan geometry of channel

Fluid properties:

ρ = density of fluid

μ = dynamic viscosity of fluid

Bed-material properties:

D = represnetative size of bed-material

ρ_s = density of bed-material

σ_b = shape of sediment distribution curve or gradation of bed-material

α_b = shape factor of bed-material

and g = acceleration of gravity.

The variables can be arranged:

$$q_s = f_1(d_*, S, C_w, b, fc, fg, \rho, \mu, D, \rho_s, \sigma_b, \alpha b, g) \quad (4.1)$$

It is possible to express certain variables in terms of others; g and S can be replaced by γ_s', the submerged specific weight of the bed-material, and v_*, the shear velocity, using the relationships:

$$\gamma_s' = g\ (\rho_s - \rho)$$

and

$$v_* = \sqrt{g d_* S}$$

The bed-load transport is therefore a function of:

$$q_s = f_1(d_*, v_*, C_w, b, fc, fg, \rho, \mu, D, \rho_s, \sigma_b, \alpha_b, \gamma_s') \quad (4.2)$$

The bed-load transport is dependent upon thirteen variables, some of which will have to be discarded if this complex phenomenon is to be dealt with. The concentration of fine suspended material C_w has a significant influence on the transport of sand (Simons, Richardson and Hauschild 1963) but little information exists on its influence on the transport of coarse bed-material. For this reason this variable will be neglected.

The channel width b has minimal influence on flow conditions if the width is more than five times the flow depth. If the width is narrower a sidewall correction can be applied; this variable can therefore be deleted.

The existing data for coarse material transport is confined to flumes or rivers having straight alignments with

reasonably regular cross-section shapes. The variables fc and fg are therefore removed.

The influence of the gradation and shape of the bed-material have not been studied in detail. It is necessary to assume that the various materials used in the flume tests were similar in their gradation and shape.

The removal of these variables results in:

$$q_s = f_2 \, (d_*, v_*, \rho, \mu, D, \rho_s, \gamma_s') \qquad (4.3)$$

Using the Buckingham π-theorem the following dimensionless parameters can be obtained (choosing ρ, D, and v_* as repeating variables):

$$\frac{q_s}{\rho v_*{}^3} = f_3 \, (\frac{d_*}{D}, \frac{\rho v_*{}^2}{\gamma_s' \, D}, \frac{\rho D v_*}{\mu}, \frac{\rho_s}{\rho}) \qquad (4.4)$$

The relative density parameter $\rho_{s/\rho}$ has been shown to be insignificant for threshold of motion conditions (Gessler 1965) and the same conclusion is assumed to hold for the transport of material. Equation 4.4 then reduces to:

$$\frac{q_s}{\rho v_*{}^3} = f_4 \, (\frac{\rho D v_*}{\mu}, \frac{\rho v_*{}^2}{\gamma_s' \, D}, \frac{d_*}{D}) \qquad (4.5)$$

For convenience the above dimensionless variables will be defined as follows:

$$P = q_{s/\rho v_*{}^3}$$

$$X = \frac{\rho D v_*}{\mu}$$

$$Y = \frac{\rho v_*{}^2}{\gamma_s' \, D}$$

$$Z = d_{*/D}$$

The most compact, although not the most convenient, form of presentation would be a three-dimensional plot as shown in FIGURE 24. If the dimensionless variable $\frac{d_*}{D}$ has a significant influence on the transport rate several "surfaces" will be evident. It would also be more convenient to change the variable X to X^2/Y which yields:

$$\frac{X^2}{Y} = \frac{\gamma s' D^3}{\rho \nu^2}$$

In flume experiments, with water temperatures kept constant, the above X^2/Y variable will depend only on the type and size of sediment. The sediment fed into the flume can be controlled at will, resulting in a constant value of X^2/Y.

The final relationship is therefore:

$$qs/\rho v_*^3 = f_5 \left(\frac{\gamma s' D^3}{\rho \nu^2}, \frac{\rho v_*^2}{\gamma_s' D}, \frac{d_*}{D}\right) \quad (4.6)$$

This relationship is identical to that of Yalin (1965) and similar to that of Cooper and Peterson (1968) who arrived at:

$$\frac{Vm^2}{gd_*} = F \left(\frac{\sqrt[3]{\nu g D}}{\nu}, C, \frac{d_*}{D}\right) \quad (4.7)$$

where: C = bed-load charge in parts per hundred thousand by weight. Cooper and Peterson (1968), however, considered C as independent and imposed on the system and obtained a Froude number to relate the intensity of flow instead of a mobility number Y.

4.3 ANALYSIS OF FLUME DATA

The data from the flume experiments for particles over 2 mm in size are tabulated in APPENDIX 4 and will serve as a basis for checking the dimensional analysis.

However, slight changes in the variables were necessary for several reasons. Side wall corrections were necessary due to the difference in surface roughness between glass or steel flume walls and a gravel bed. The correction techniques initially developed by Einstein (1942) and Johnson (1942) and modified by Vanoni and Brooks (1957) were used in analyzing the available data. This sidewall correction resulted in a change in the hydraulic radius R which is now designated as the hydraulic radius of the bed R_b; the friction factor f becomes the friction factor of the bed f_b and the shear velocity v_* now pertains to the bed and is noted as v_{*_b}. The various dimensionless variables now become:

$$Y = \frac{\rho v_{*b}^{2}}{\gamma_s' D}$$

$$Z = R_b/D \qquad (R_b \text{ replaced } d_*)$$

$$\frac{X^2}{Y} = \frac{\gamma_s' D^3}{\rho \nu^2}$$

and $$P = \frac{q_s}{\rho v_{*b}^{3}}$$

The first group of plots of P versus Y for various values of X^2/Y, or in effect for different gravel sizes, are

shown in FIGURES 25 to 34 and represent flume tests with bed-material of uniform size. These plots indicate that the relative depth Rb/D has a distinct influence on the transport of the coarse-bed material. Lines having equal values of Rb/D ranging from 10 to 30 were established on these figures. The effect of increasing the depth and consequently the value of Rb is to decrease the rate of material in transport. It is apparent that doubling the flow depth, for one sediment size and the same shear velocity, can result in a reduction of the bed-load transport by a factor of four or five.

These ten plots, from FIGURES 25 to 34, all pertain to uniform material and a subsequent plot of P versus Y for a constant value of Rb/D = 15 (FIGURE 35) indicates that the parameter X^2/Y or the particle size is also significant. This plot shows that the relation between the mobility number Y (or the flow intensity) and the sediment transport parameter P is rather complex; it would be difficult to develop equations relating these parameters since the resultant plot shows a three dimensional curved surface. A similar plot (FIGURE 36) for a different relative depth value will yield a surface in a slightly different position from the first plot and if more data existed a family of surfaces could be obtained. This family of surfaces should describe the phenomena completely for the sizes of material used in the flume tests, however, these surfaces may not be

applicable to coarse-bed river since the range of particle sizes found on the beds of these rivers is very wide. Surfaces developed from flume tests using bed-material mixtures would be required in this case.

In this regard, data from flume tests by Meyer-Peter and Muller and other investigators are plotted in FIGURES 37 to 43. The same type of plots with lines of constant relative depth are obtained, only in this case the surfaces are higher than in the previous plots for uniform materials. The data by Liu and Carter, however, plot rather inconsistently - this is not surprising as some of the tests from this series show the sidewalls of the flume to be rougher than the bed. These tests exhibit low shear velocity values which would be the result of errors in either slope or water depth.

FIGURE 44 shows a portion of a surface for a relative depth equal to 50, but the data are sparse yielding a very small portion of the desired surface. This plot illustrates conclusively the fact that many more flume experiments are necessary and shows where the data are lacking.

A direct comparison of the derived surfaces for both uniform and mixed bed-material, for relative depth values of 20, is shown in FIGURE 45. This plot indicates that there is some distinction between transport of bed-material having a mixture of sizes as compared to uniform

material when the flow intensity Y is identical. The data are, however, too sparse to make any definite conclusions.

It may be in order, at this stage, to compare the derived dimensionless plots to commonly used bed-load transport formulas. The Einstein bed-load function (Einstein 1950) is the most popular in North America, and relates the bed-load function Φ to the intensity of shear ψ:

$$\Phi = F\ (\psi)$$

which is:

$$\frac{q_s}{\gamma_s'} \left(\frac{\rho}{\rho_s}\right)^{1/2} \left(\frac{1}{gD^3}\right)^{1/2} \times \frac{1}{F} = \frac{\gamma_s' \quad D_{35}}{\gamma \ R_b' \ S} \qquad (4.8)$$

where: R_b' = hydraulic radius with respect to particle.

$$F = \sqrt{2/3 + \frac{36\nu^2}{gD^3(S_s-1)}} - \sqrt{\frac{36\nu^2}{gD^3(S_s-1)}}$$

S_s= specific gravity of particles.

Chien (1954) has also shown that the Meyer-Peter and Muller bed-load transport equation is almost identical to that of Einstein which should not be surprising since both are derived from essentially the same flume data.

The ψ function of Einstein (1950) is the reciprocal of Y ($\psi = \frac{1}{Y}$) whereas the bed-load function $\phi = P\frac{V_*}{w}$, where w is fall velocity (Yalin 1966). The ratio $^{v*}/_{w}$ is a function of X and Y which means that Einstein's relationship took into account three of the four dimensionless parameters; the effect of relative depth was not considered. The curves

shown in FIGURES 35 and 36 indicate that the effect of relative depth Z is significant; the curves in FIGURE 35 define a surface for $Z = {}^{R}b/_{D} = 15$ which is somewhat higher than the surface for Z = 20 in FIGURE 36. Ignoring the Z parameter and plotting P versus Y, for the uniform material, yields a graph with a fair degree of scatter (FIGURE 46). No attempt was made to obtain the best fit line through the plotted points as the derived equation would not include all the significant variables. A comparable degree of scatter is to be found on a plot of Einstein's bed-load function (FIGURE 47) as produced by Laursen and Rouse (Brown 1949). Instead of attempting to arrive at one functional relationship relating bed-load transport to flow intensity it would be more realistic to develop design charts for P versus Y taking into account Z and the effect of particle size $({}^{X^2}/_{Y})$. A program to investigate similar parameters is underway at the University of Alberta (Cooper and Peterson 1968, Cooper 1970) using all available sediment transport data (sand and gravel) and has shown reasonable success to date.

4.4 DESIGN CURVES FOR SEDIMENT TRANSPORT OF COARSE MATERIAL

Design engineers, in general, prefer to use design curves that are not dimensionless. The dimensionless plots discussed previously are assumed to apply to all types of materials as well as fluids, however, if only coarse material under the action of water is considered the plots can

be simplified.

Colby (1964 a,b) briefly points out that the bed-material discharge in natural rivers can be related to four common measures:

1. Total shear on the stream bed, $\tau_o = \gamma RS$
2. Mean velocity, V_m
3. Shear velocity, $v_* = \sqrt{gRS}$, and
4. Stream power, $\tau_o\ V_m$.

Colby compares these various measures and concludes that the relationship of bed-material discharge to mean velocity is the most convenient to apply. Using the mean velocity does not require a knowledge of the energy slope which is generally difficult to obtain with high accuracy.

With this in mind the flume data from the previously quoted investigators was used to obtain FIGURES 48 to 55 for uniform materials and FIGURES 57 to 60 for bed-material mixtures. Each figure shows the unit bed-load discharge q_s varying with the mean velocity V_m for one representative particle size and a range of relative depths. The plots indicate that for a constant velocity the bed-load transport will decrease as the water depth (hydraulic radius) increases.

FIGURE 56 shows a partial three-dimensional surface with the relative depth R_b/D equal to 15 and the uniform particle size being the third dimension. More data are required to give a complete surface. This plot, however,

indicates that for a constant velocity and relative depth the amount of uniform material transported is dependent on the size of the material; the smaller the particles the larger the volume transported in a definite time period. This would mean that the transport capacity of a stream is reduced if coarse material is dumped into it, even if all the dumped material is above the threshold state.

In order for the flume data to be useful in estimating bed-load discharge in rivers similar plots would have to be derived for bed-material mixtures. However, as shown in FIGURES 57 to 60 the range of data is insufficient; the range of particle sizes used in the experiments varied from 2.0 to 4.4 mm.

At the moment design curves applicable to natural rivers cannot be derived due to a lack of adequate data. Laboratory investigations using bed-material mixtures as well as field investigations covering a wide range of representative particle sizes and relative depths are a necessity.

4.5 COMPARISON OF EXISTING BED-LOAD FORMULAS TO FIELD MEASUREMENTS

To date there are very few data relating the bed-load transport of a coarse-bed river to its hydraulic characteristics. The procurement of this data is extremely difficult as large and cumbersome bed-load traps are required

to catch moving stones. Recently, however, bar movements on the North Saskatchewan River at Drayton Valley were recorded (Galay 1967a) from which estimates of bed-load transport can be made. Also, the bed-load transport in the Elbow River in Alberta was studied in some detail (Hollingshead 1968 a,b). These cases will now be discussed in more detail.

4.5(a) <u>North Saskatchewan River at Drayton Valley</u>

During the summer of 1965 successive soundings of the river-bed yielded the bed forms shown in FIGURE 61 and PHOTOGRAPH 10. Soundings along the west line indicate that several large bars shifted downstream by several hundred feet during a four-day sounding time interval (June 15 to June 19). The flow conditions during these soundings were:

Discharge-25,000 to 43,000 cfs, (average 35,000 cfs)
Mean depth - 10 feet
Water surface width - 775 feet
Slope - 0.0015
Representative bed-material size - 1.1 inches

Considering a one-foot strip the following volumes of coarse-bed material were moved during the four day period:

Upstream bar from x-sec. 89 - 690 cu. ft.

Downstream bar from x-sec. 89 - 1330 cu. ft.

Mean volume moved - approximately <u>1,000 cu. ft.</u>

This volume corresponds to 1000 x 105 - 105,000 lbs. of dry bed-material. The steady rate of bed-material transport required to supply this amount of material over a four

day period would be:

$$\frac{105,000}{4x24x60x60} = 0.31 \text{ lb./ft. sec.}$$

A comparison of this transport rate, which is in itself relatively approximate, to the rate obtained by various bed-load formulas is shown below:

NORTH SASKATCHEWAN RIVER
BED-LOAD TRANSPORT RATES (LB./FT.SEC.)
(x-sec. 89, Q = 35,000 cfs)

BAR MOVEMENT TECHNIQUE	BLENCH	COOPER & PETERSON	MEYER-PETER & MULLER
0.31	0.48	0.28	0.41

The Cooper-Peterson (1968) computation yields a result relatively close to that computed from the bar movement, however, it is difficult to know whether the bar transport rate is representative for the cross-section. Appendix V shows the details of the bed-load computation using Blench's regime equations.

4.5(b) Elbow River at Bragg Creek, Alberta

A number of bed-load discharge measurements were taken during the summer of 1967 using basket and VUV samplers (Hollingshead, 1968 a,b). The results of the samplings along with the hydraulic characteristics of the river are shown in TABLE 7.

The bed-load transport q_s is plotted against the mean velocity V_m with the relative depth as the third

variable (FIGURE 62). Although the data are sparse a line having d_*/D equal to 30 was sketched in. This line appears to have a similar slope and curvature to the curves based on coarse bed-load transport in flumes. It is, however, impossible to add more relative depth lines to this graph - more data is required.

A comparison of the measured transport rate in the Elbow River to the rates computed from various bed-load formulas is shown in TABLE 8.

It is difficult to compare the computed transport rates with the measured values by viewing the table. There fore, the computed transport rates were plotted versus the mean flow velocity and curves having a relative depth d_*/D' equal to 30 were determined for each bed-load transport formulas. These curves were replotted in FIGURE 63 along with the measured transport rate. It is apparent that there is no single transport formulas that can be recommended over all the rest; the curves are based on flume experiements using uniform sand as the bed material. The charts developed by Cooper and Peterson (1968) are, however, the easiest to use; one of the charts is reproduced in FIGURE 64. The extrapolation of the "sand-bed formulas" to coarse-bed rivers is, however, not the sole reason for wide

TABLE 7

BED-LOAD TRANSPORT IN THE ELBOW RIVER AT BRAGG CREEK, ALBERTA
(at Cableway, Sta. 12+18 D/S)

D_{50} = 0.083 feet

DATE	Q cfs	b_w ft.	$d_*=A/b_w$ ft.	S	V_m ft./sec.	q_s lb./ft. sec.	d_*/D_{50}
1967							
June 1	3850	160	2.94	0.00745	8.40	0.855	35
June 2	2900	150	2.68	0.00745	7.40	0.540	32
June 17	1900	136	2.30	0.00745	6.20	0.304	28
June 19	1900	136	2.30	0.00745	6.20	0.310	28
June 20	1630	131	2.20	0.00745	5.80	0.092	26
June 21	1560	130	2.17	0.00745	5.66	0.197	26
June 22	1550	130	2.16	0.00745	5.65	0.086	26
June 22	1470	128	2.12	0.00745	5.50	0.175	26
June 23	1300	124	2.03	0.00745	5.20	-	24
1968							
June 8	1490	129	2.13	0.00745	5.45	0.196	26
June 9	1400	127	2.09	0.00745	5.30	0.014	25
June 10	1370	126	2.05	0.00745	5.26	0.023	25

TABLE 8

BED-LOAD TRANSPORT COMPARISONS
ELBOW RIVER

(Sta. 12+18)

(lb./ft. sec.)

Q cfs	Measured q_s	Meyer-Peter q_s	Blench q_s	Cooper & Peterson q_s
1,000	-	-	0.080	-
1,300	-	0.177	0.104	-
1,470	0.125	0.330	0.122	0.011
1,550	0.061	0.424	0.126	0.015
1,630	0.066	0.540	0.132	0.027
1,900	0.217	0.875	0.157	0.105
2,900	0.387	2.270	0.240	1.090
3,850	0.610	3.800	0.314	2.250
5,000	-	5,760	0.403	3,570

divergence of the curves in FIGURE 63. Using these formulas on sand-bed rivers results in curves that also diverge markedly from each other (FIGURE 65). Only Blench's formula makes an allowance for the geometry of the channel in plan (fg).

4.6 CONCLUSIONS

The solutions to important questions dealing with reservoir sedimentation and channel degradation on coarse-bed rivers depend upon an accurate assessment of the bed-load transport. However, dimensionless plots based on coarse bed-load transport in flumes indicate that there is insufficient data to develop relationships that would be applicable to natural rivers. The wisdom in attempting to arrive at one relationship that could be used in computing bed-load transport is questioned; dimensional analysis has shown that the transport is related to at least three dimensionless parameters.

A comparison of computed bed-load transport using several common formulas and charts (Meyer-Peter and Muller, Blench, Cooper-Peterson) to the actual measured transport in the Elbow River indicates that no one formula can be recommended over all others. The Meyer-Peter and Muller formula is generally recommended for coarse-bed rivers; however, applying it to the Elbow River resulted in transport values 6 or 7 times the actual measured values. Of the formulas

and relationships examined it was found to be the least reliable. The relationships developed by Cooper and Peterson (1968) were the easiest to use. Most of the formulas were based on data from sand-bed channels and flumes and yield widely varying answers even for channels in sand. Computing the bed-load transport, in the North Saskatchewan River, from shifting gravel bars yielded transport rates that compared favourably with computations based on various formulas.

4.7 RECOMMENDATIONS

Flume study data for coarse bed-material mixtures are virtually non-existent. This data would be useful in understanding the relationship between transport rate and significant variables. The tests should utilize a well graded mixture having a representative diameter of at least 20 mm. and should be conducted over a wide range of depths. With conventional flumes a flow depth of three feet would be the maximum possible, yielding a relative depth of 40. Larger ratios of relative depth would be desirable. Mixtures of light weight materials would also yield information on the importance of the density of the transported material.

The collection of data from mobile bed rivers should coincide with and complement the flume studies recommended above. More transport measurements should be conducted on rivers similar in size to the Elbow River (relative depths varying from 20 to 50). Investigations on larger rivers such as the North Saskatchewan and the Athabasca would yield information on rivers having relative depths up to 150. The use of tracer techniques may be necessary in these larger rivers as the trapping of coarse material would be difficult. Continuous sounding of moving dunes and major bars would yield valuable information on the rate of transport as well as the form of bed roughness.

The plotting of data in a direct manner such as transport rate versus mean velocity with relative depth as the third variable appears promising. Plots of this type would be readily used by practicing engineers as they are direct and uncomplicated. Effort should be directed towards establishing the most practical plots since there is much discrepancy among existing complicated formulas.

CHAPTER V

RESISTANCE TO FLOW IN COARSE-BED RIVERS AND FLUMES

5.1 INTRODUCTION

In alluvial channels the bed forms generated during the passage of high flows are complex and are continuously undergoing change. Laboratory experiments (Simons and Richardson, 1961; Guy, Simons and Richardson, 1966) have established that the bed forms in sand-bed rivers pass through the following regimes (see FIGURE 66):

(a) Plane bed with no sediment motion

(b) Ripples

(c) Ripples on dunes

(d) Dunes

(e) Washed out dunes

(f) Plane bed with motion

(g) Standing waves

(h) Anti-dunes

(i) Chutes and pools.

Needless to say, these changes in the bed forms would produce appreciable changes in flow resistance.

All the experiments and observations upon which this bed-form classification is based involve sand as the alluvial material. The extrapolation of this classification to coarse-bed alluvial channels may, at the moment, be

unwarranted. Echo-soundings on the North Saskatchewan River (Galay, 1967b) have shown that dunes in coarse-bed rivers behave differently from dunes in sand-bed rivers. The main differences noted were:

(1) The wave length decreased with an increase in flow depth and velocity which is the opposite for dunes in sand-bed channels (see PHOTOGRAPH 10);

(2) The dune amplitude did not appear to be controlled by the depth of flow. The amplitude of dunes in sand-bed channels appears to increase directly with depth (Nordin and Algert, 1965).

A similar phenomenon was noted by Theil (1932) who described bed forms exposed on the bed of a stream that partially drained a lake in Minnesota. The wave lengths varied from 25 to 60 feet and were found to decrease in length as the velocity increased. Theil, however, erroneously assumed that the velocity decreased in a downstream direction.

Flume experiments by many researchers have also verified that coarse bed-material will not form into ripples. Simons and Richardson (1966) have produced a graph, shown in FIGURE 67, which shows that ripples do not develop if the bed-material is larger than 0.65 mm. Photographs of water surface features suggesting the presence of antidunes in a coarse-bed river in New Zealand have also been presented by Thompson (1963). The range of bed forms that may be generated

are open to investigation.

The fact remains that the hydraulic engineer is called upon to evaluate the roughness or the resistance to flow of a coarse river-bed in either an immobile or mobile state. This chapter will assess the resistance of coarse river beds in two distinct states - when the river-bed is immobile and can be considered as a rigid bed and when the bed is fully mobile.

5.2 IMMOBILE RIVER-BED

5.2(a) Dimensional Analysis

The resistance to flow over a rigid coarse bed depends upon the flow conditions, the channel geometry, the properties of the fluid and the characteristics of the protrusions on the boundary. The variables involved can be assembled in the following manner (see FIGURE 68):

Flow conditions

V_m = mean flow velocity

d_* = mean flow depth

S = slope of energy gradient

C_w = concentration of suspended particles

Channel geometry

b = width of channel

f_c = factor defining cross-sectional shape of channel

f_g = factor defining plan geometry of channel

<u>Fluid properties</u>

ρ = density of fluid

μ = dynamic viscosity of fluid

<u>Protrusion characteristics</u>

k = height of protrusion (usually k_{50})

α_b = shape factor of protrusions

λ = concentration of protrusions

χ = areal pattern of protrusions

and g = acceleration of gravity

Setting this out in equation form:

$$f_1\,(V_m, d_*, S, C_w, b, f_c, f_g, \rho, \mu, k, \alpha_b, \lambda, \chi, g,) = 0 \qquad (5.1)$$

Several of the above variables can be omitted in order to simplify the equation. The data to be analyzed have been derived from straight flumes or rivers having a uniform wide cross-section (width more than $5d_*$); the variables b, f_c, and f_g are therefore eliminated. The effect of flow concentration C_w on resistance involving large protrusions on the bed has not been investigated and will be ignored in this treatment. The protrusion or particle shape factor α_b does not vary from river to river as shown in FIGURE 69 and is tentatively eliminated. The effect of the concentration and distribution of bed particles on resistance is difficult to assess directly in natural rivers and will be tentatively ignored.

Equation 5.1 therefore reduces to:

$$f_2 \ (V_m, d_*, S, \rho, \mu, k, g) = 0 \qquad (5.2)$$

According to the Buchingham π theorem the above variables can be reduced to four dimensionless parameters. Choosing ρ, V_m and d_* as repeating variables results in:

$$f_2 \left(\frac{V_m}{\sqrt{gd_*}}, S, \frac{\rho V_m d_*}{\mu}, \frac{d_*}{k}\right) = 0 \qquad (5.3)$$

For any significant flow in coarse-bed rivers the flow can generally be described as fully rough turbulent flow which makes the Reynolds number $\frac{\rho V_m d_*}{\mu}$ of secondary importance. Therefore:

$$f_3 \left(\frac{V_m}{\sqrt{gd_*}}, S, \frac{d_*}{k}\right) = 0 \qquad (5.4)$$

This equation can now be investigated in the above form, however, replacing S by τ_o through the relationship $\tau_o = \gamma d_* S$ leads to a familiar resistance term $\frac{V_m}{v_*}$ or $\frac{C}{\sqrt{g}}$ where C = Chezy's C:

$$f_4 \left(\frac{V_m}{\sqrt{gd_*}}, \frac{V_m}{\sqrt{\tau o/\rho}}, \frac{d_*}{k}\right) = 0 \qquad (5.5)$$

or re-arranging:

$$\frac{V_m}{v_*} = f_4' \left(\frac{V_m}{\sqrt{gd_*}}, \frac{d_*}{k}\right)$$

The Froude number $V_m/\sqrt{gd_*}$ assumes importance only when appreciable surface waves develop (Rouse, Koloseus and Davidian 1963) and is eliminated since most of the data to be analyzed were for Froude numbers less than one. This

results in:

$$\frac{V_m}{v_*} = f_5 \left(\frac{d_*}{k}\right) \tag{5.6}$$

5.2(b) Emperical Flow Formulas

Instead of using the concentration and pattern parameters explicitly, investigators have taken the protrusion height k to be larger than the median size of the bed-material and assumed that this will account for these unknown parameters. Kellerhals (1963) arrived at the following formula:

$$\frac{V_m}{v_*} = 6.5 \left(\frac{d_*}{k}\right)^{1/4} \tag{5.7}$$

where: $k = D_{90}$ from a grid sample analyzed by number.

Kellerhals' formula is almost identical to the logarithmic flow formula formulated by Keulegan (1938) for wide channels:

$$\frac{V_m}{v_*} = 5.75 \log\left(\frac{d_*}{k}\right) + 6.00 \tag{5.8}$$

In this formula the k parameter refers to a uniform size of bed-material and is based on experiments carried out by Nikuradse (1932) who coated the inside of pipes with uniform sand and conducted tests to assess the effect of this sand roughness on flow. However, it is apparent that the protrusion height of the glued sand particles would be some fraction of the particle diameter, possibly equal to one-half the diameter as the grain contact points would be at D/2 from the boundary.

It has also been noted by several investigators (Lane and Carlson 1954) that coarse bed-material is generally oriented into a "shingle pattern" on a river-bed. PHOTOGRAPH 11 illustrates this shingle pattern and FIGURE 70 shows that the particles will orient themselves so that the long "a" axis is perpendicular to the flow. It is apparent that the protrusion height of the bed particles into the flow may not be the intermediate b axis as used by some investigators. The question remains, however, what measure should be used as the protrusion height?

A "bed roughness meter" was therefore constructed to directly measure the protrusion height of gravel and cobbles on a shingled stream bed. PHOTOGRAPH 12 shows the "roughness meter" in operation on the North Saskatchewan River. A movable arm is drawn over a cobble bed with its vertical motion recorded on a moving chart. The chart and arm movements were geared so that the resulting trace dimensions were exactly one-half the actual bed pattern dimensions. A sample of the resulting trace is shown in FIGURE 71. Measurements of the three axes of cobbles were also taken at the same location as the protrusion height measurements.

From the relatively few measurements of protrusion height taken in 1966 and 1967 a plot of median protrusion height versus median intermediate (b) and median minor (c)

axis was obtained (FIGURE 72). A tentative conclusion from this plot would be:

Protrusion height $k = 1.0c$

or Protrusion height $k = 0.67b$ or $0.67D_{50}$ (by number)

This information is now used to obtain flow formulas for coarse-bed rivers which would be based on the projection height of the bed-material into the flow.

Data from coarse-bed rivers having distinctly immobile beds was gathered and tabulated in TABLE A-7, APPENDIX 6. A plot of $\frac{Vm}{v_*}$ versus d_*/k results in the equation (FIGURE 73):

$$\frac{Vm}{v_*} = 3.0 \left(\frac{d_*}{k}\right)^{0.45} \qquad (5.9)$$

The plot exhibits a fair degree of scatter; the points that deviate markedly are from investigations reported by Barnes (1967) and Alberta Water Resources. The reason for the scatter may be the variation in residual bed forms or bars that exist when the bed material stops moving.

The power and constant in equation 5.9 do not agree with the values in Kellerhals' equation 5.7, but the protrusion height may not be directly related to the D_{90} by number. For this reason flume tests on naturally sorted gravel beds were carried out. A description of the tests along with their results is presented in APPENDIX 7. The flume tests with the natural gravel bed plot very closely

to the derived line (equation 5.9) as shown in FIGURE 74. The data, however, covers a limited range of relative depths.

Further flume tests were carried out having an artificial cemented gravel bed similar to Nikuradse's sand pipe tests (see PHOTOGRAPH 13). It is interesting to note that the results from the cemented gravel bed indicate higher roughness values than the natural sorted gravel bed even though the median projection height was higher for the natural bed (FIGURE 74). The orientation of the gravel by flowing water, therefore, has a significant effect on the resistance to flow. Therefore, the extension of Nikuradse's pipe tests with cemented sand grains to natural rivers would be highly questionable.

The power formula (equation 5.9) can be replaced by a logarithmic formula:

$$\frac{Vm}{v_*} = 8.0 \log(\frac{d_*}{k}) + 1.0 \qquad (5.10)$$

This equation is of the form proposed by Keulegan (1938) and Robinson and Albertson (1952):

$$\frac{Vm}{v_*} = \frac{2.30}{K} \log (\frac{d_*}{k}) + C_1$$

where: K = vonKarman turbulence coefficient

C_1 = constant

The logarithmic formula, 5.10, would have a turbulence coefficient K equal to 0.29 instead of the commonly accepted 0.40. The turbulence coefficient does range from 0.20 to

to 0.40 for sediment laden flows (Chien 1956) and may vary in rigid boundary flow due to such factors as secondary circulation and boundary roughness. Equation 5.10 along with Keulegan's equation 5.8 are shown in FIGURE 75. The use of Keulegan's equation implies the acceptance of a coefficient and constant which are based on Nikuradse's pipe tests and results in a computation of an "equivalent sand grain size". A more realistic approach is, however, suggested here - the protrusion height that is actually measured should be used in a flow formula with the turbulence coefficient and constant adjusted to fit the observed flow conditions. This approach results in the use of actual protrusion heights; flume experiments utilizing artificial roughness elements such as bars (Powell 1946), baffles (Sayre and Albertson 1963), and cubes (O'Loughlin and MacDonald 1964) all used the protrusion heights in subsequent analysis.

A number of studies have recently been conducted on the resistance to flow of streams having "large bed elements", (Judd and Peterson 1969, Herbich and Shulits 1964). Uniform two-dimensional flow was not always present for the tabulated flows as the bed elements were so large that they represented changes in cross-section rather than surface texture. For this reason the data were not included in the analysis dealing with immobile channels.

5.2(c) Manning's Roughness Coefficient

In North America the use of the Manning equation to compute flow in natural channels is widespread. Many engineers can associate the physical appearance of a stream-bed with a Manning's roughness coefficient n. The roughness coefficient could also be estimated by using the Strickler formula:

$$n = 0.032\ D^{1/6} \tag{5.11}$$

with D being D_{50} and measured in feet.

A better relationship for estimating Manning's n which would include the effects of flow depth and make use of the protrusion height of the bed-material can be obtained by using the Manning equation with the previously derived power flow equation (Equation 5.9):

$$Vm = \frac{1.49}{n}\ d_*^{2/3}\ S^{1/2} \tag{5.12}$$

and using:

$$\frac{Vm}{v_*} = 3.0\ \left(\frac{d_*}{k}\right)^{0.45}$$

yields:

$$\frac{n}{d_*^{1/6}} = \frac{0.088}{\left(d_*/_k\right)^{0.45}} \tag{5.13}$$

where k = 0.67 D_{50} in feet.

The data from APPENDIX 6 have been plotted with $n/_{d_*^{1/6}}$ versus $d_*/_k$ in FIGURE 76. There is again a fair degree of scatter, but the above derived equation does appear

to pass through the points reasonably well; this relationship would, therefore, yield a reasonable estimate of the Manning coefficient for a reasonably straight immobile coarse river-bed. Its use, however, should be limited to flows below those that would cause most of a rivers' bed to be in motion; this aspect of resistance to flow with a mobile stream bed becomes fairly complex.

5.3 MOBILE RIVER-BEDS

5.3(a) Dimensional Analysis

As in the chapter dealing with the transport of coarse bed-material (CHAPTER IV) the resistance to flow is related to the flow conditions, the channel geometry and the properties of the fluid and bed-material. The mean flow velocity is therefore a function of:

$$Vm = f_1(d_*, S, C_w, b, f_c, f_g, \rho, \mu, D, \rho_s, \sigma_b, \propto_b, g) \qquad (5.14)$$

The following variables C_w, b, f_c, f_g, σ_b, and $\propto_b$ are now deleted for the reasons stated in CHAPTER IV, section 4.2.

This results in:

$$Vm = f_2\ (d_*,\ S,\ \rho,\ \mu,\ D,\ \rho_s,\ g) \qquad (5.15)$$

It is possible to express certain variables in terms of others; g and S can be replace by γ_s' and v_* using the relationships:

$$\gamma_s' = g\ (\rho_s - \rho)$$

and

$$v_* = \sqrt{g d_* S}$$

This changes equation 5.15 to:

$$Vm = f_2\ (d_*, v_*, \rho, \mu, D, \rho s, \gamma_s') \qquad (5.16)$$

Using the Buckingham π theorem with ρ, D, and v_* as repeating variables results in:

$$\frac{Vm}{v_*} = f_3\ \left(\frac{\rho D v_*}{\mu}, \frac{\rho v_*^2}{\gamma_s' D}, \frac{d_*}{D}, \frac{\rho s}{\rho} \right) \qquad (5.17)$$

Assuming again that the relative density parameter is of minor importance, that d_* and v_* are replace by R_b and v_{*b} respectively and that the particle Reynolds number can be manipulated leads to:

$$\frac{Vm}{v_{*b}} = f_4\ \left(\frac{\gamma_s' D^3}{\rho \nu^2}, \frac{\rho v_{*b}^2}{\gamma_s' D}, \frac{R_b}{D} \right) \qquad (5.18)$$

5.3(b) Analysis of Flume Data

Plots of $Vm/_{v_{*b}}$ versus the mobility number $\frac{\rho v_{*b}^2}{\gamma_s' D}$ for various values of $R_b/_D$ and for several sizes of material are shown in FIGURES 77 to 81. These plots all pertain to uniform material. A subsequent plot of $Vm/_{v_*}$ versus the mobility number for a constant value of $R_b/_D = 15$ (FIGURE 82) indicates that the size of material has little influence on resistance. This was, however, not the case for sediment transport. The amount of uniform material in transport, with $R_b/_D$ and the mobility number constant, varies with the size of material (see FIGURE 35), but the resistance to flow is not greatly affected. The resistance function $Vm/_{v_*}$ is reasonably constant at a value of 10.5 for mobility numbers ranging from 0.10 to 0.20.

The data for uniform materials as obtained by Bogardi and Yen are shown in FIGURES 83 to 85, but the range of flow depths used was very limited. The same can be said of the data from flume tests using bed-material mixtures (FIGURES 86 to 89); the data are very sparse and relative depth curves are difficult to determine. However, the resistance for bed-material mixtures is higher than for uniform materials, for the same mobility number (FIGURE 90). These findings are contrary to the conclusions obtained by Daranandana (1962) who studied the effect of the gradation of sands on flow phenomena. He found that the resistance to flow was considerably greater for a uniform sand as compared to a poorly sorted sand when the bed forms were in the dune phase. This would imply that conclusions derived from flume experiments using sands cannot be extrapolated to coarse-bed materials.

5.3(c) Analysis of Field Data

To date there exists little information on the resistance to flow for coarse-bed rivers under relatively large flows. Data from the Elbow River (Hollingshead 1968) and the North Saskatchewan River (Galay 1967a) are, however, available for analysis.

A plot of Vm/v_{*b} versus the mobility number for the two rivers is shown in FIGURE 91, with the data presented in TABLE 9. A curve for a rigid coarse-bed channel, based on

TABLE 9

RESISTANCE TO FLOW DATA
MOBILE CHANNELS

River	Location	Q cfs	Mean Velocity Vm f/s	Mean Particle Size D_{50} (ft.)	Slope	Mean Flow Depth d_* (ft.)	d_*/D	v_* (ft./sec.)	$\frac{\rho v_*^2}{\gamma_s D}$	Vm/v_*
Elbow	12+18 D/S	800	3.90	0.10	0.00745	2.14	21.4	0.717	0.095	5.4
(Hollingshead)		15,000	13.00	0.10	0.00745	6.60	66.0	1.260	0.292	10.3
1968a)		1,630	5.52	0.10	0.00745	2.85	28.5	0.827	0.126	6.7
		1,665	5.48	0.10	0.00745	2.92	29.2	0.836	0.129	6.5
		1,670	5.21	0.10	0.00745	2.52	25.2	0.776	0.111	6.7
		1,751	5.81	0.10	0.00745	2.90	29.0	0.835	0.129	7.0
		1,496	6.07	0.10	0.00745	2.41	24.1	0.760	0.107	8.0
		1,297	5.55	0.10	0.00745	2.28	22.8	0.740	0.101	7.5
		410	4.95	0.10	0.00745	0.93	9.3	0.472	0.041	10.5
1968b		1,490	6.0	0.10	0.00745	2.4	24.0	0.760	0.107	7.9
		1,400	5.8	0.10	0.00745	2.4	24.0	0.760	0.107	7.6
		1,370	5.7	0.10	0.00745	2.3	23.0	0.740	0.101	7.7
North Sask.	#89	23,000	4.4	0.09	0.0015	6.90	7.7	0.578	0.068	7.6
River		43,000	6.0	0.09	0.0015	9.20	10.2	0.666	0.091	9.0
(Galay, 1967a)		58,000	7.8	0.09	0.0015	9.20	10.2	0.666	0.091	11.7

equation 5.9, is also shown on the plot. The plotted points indicate that the resistance increased sharply when the material started to move and then decreased gradually as the mobility number increased. This would suggest that the bed-material initially formed into dunes which were subsequently washed out as the flow intensity increased. The North Saskatchewan River data falls into the same region as the Elbow River data but covers a smaller range of mobility numbers.

Due to the sparseness of the data it is difficult to recommend a formula or a procedure to obtain the resistance to flow in coarse mobile bed rivers.

5.4 CONCLUSIONS

The generation of complex bed forms in coarse-bed rivers makes the prediction of the resistance to flow exceedingly difficult. The problem was dealt with by separating the flow condition into two distinct categories, the first being flow over a rigid immobile bed while the second consisting of flow over a highly mobile bed.

Dimensional analysis was used to reduce the number of variables for the immobile bed condition and subsequent plotting resulted in the following equation:

$$\frac{Vm}{v_*} = 3.0 \left(\frac{d_*}{k}\right)^{0.45} \qquad (5.9)$$

The protrusion height k in the above equation is the actual protrusion of stones into the flow, not the median of the intermediate axis. This value was determined with the use

of a "roughness meter" which provided a graphical trace of the river-bed. The projection height was correlated to the intermediate diameter of the surface stones yielding:

$$\text{Projection height, } k = 0.67D_{50} \text{ (by number)}$$

Flume tests were also conducted to test equation 5.9 and the concept of protrusion height. The tests consisted of water flowing over a naturally sorted gravel bed and the results, covering a small range of relative depths, agreed well with the derived equation.

Equation 5.9 was also combined with the Manning equation to yield:

$$\frac{n}{d_*^{1/6}} = \frac{0.088}{(d_*/k)} \; 0.45$$

which is useful for estimation of Manning's roughness coefficient.

Definite conclusions regarding resistance to flow with highly mobile beds cannot be made at this date. The data are too sparse. Flume data, for uniform materials, tentatively indicate that the size of material in motion has little influence on resistance. The amount of material in transport, with relative depth and mobility number constant, varies with the size of material but the resistance to flow is not greatly affected. A comparison of uniform materials and mixtures also indicates that the resistance to flow is greater for mixtures at comparative flow conditions, more

flume data are required to verify these findings.

Limited field data suggest that the bed-material, once in motion, forms into dunes and then into a plane bed as the flow intensity increases. The resistance function Vm/v_* decreases by a factor of two (indicating added resistance) as soon as the material is in motion. The use of a rigid bed flow formula in highly active coarse-bed rivers can lead to large errors in flow estimation. However, no formula or procedure was recommended for use under these types of flow conditions.

5.5 RECOMMENDATIONS

The recommendations presented in the chapter dealing with sediment transport (CHAPTER IV) would also be applicable to this chapter. The material on the bed of a river is placed into motion by the adjacent flow which then has its turbulence characteristics modified by the moving bed-material; the movement of the bed-material, the resistance to flow and the motion of the fluid are inter-dependent.

The experiments conducted with flows over a naturally sorted rigid gravel bed should be extended - data having relative depths up to 100 would be valuable in establishing a rigid bed flow formula. The variation of protrusion height with the shape of stones on the bed of an immobile channel should also be investigated.

The range of bed forms developed in sand-bed channels undergoing large changes in magnitude of flow have been investigated extensively. However, the type and sequence of bed forms generated in coarse-bed channels has not received the same attention; flume studies using bed-material mixtures under a wide range of relative depths (up to 50) and mobility numbers would be necessary. The variation in the resistance function ($^{Vm}/_{v_*}$) with the types of bed forms being generated would be clarified and relationships for the prediction of flow resistance parameters would be forthcoming.

The collection of data from mobile bed rivers, such as the Elbow River, should also continue. Echo soundings along with appropriate flow measurement would yield information on the development of bed forms and their effects on flow resistance.

CHAPTER VI

THE DESIGN OF STABLE CHANNELS IN COARSE MATERIALS

6.1 INTRODUCTION

The design of river diversions or irrigation canals in coarse bed-material has not received the attention that sand-bed channels have to date. The design of a channel that will be comparatively stable is of major importance in the proper layout of vast river diversions and irrigation schemes such as those contemplated by the Saskatchewan-Nelson Basin Board (1969).

This chapter will briefly present aspects of dominant and bankfull discharge and relate bankfull discharge to stable widths and depths. Design equations are presented for the design of stable widths, depths and slopes. A tentative chart is presented from which an assessment of bankfull discharge can be made knowing flow conditions at some low stage of flow.

6.2 DOMINANT AND BANKFULL DISCHARGE

The hydraulic geometry of the river channel, namely the width, depth and slope, depend primarily upon the flow in the channel (Lacey 1929, Blench 1969). In order to design channels it is necessary to obtain relationships between the hydraulic geometry and the flow characteristics.

The main question at this point is - what discharge should be used in attempting to arrive at meaningful relationships? Leopold and Maddock (1953) have used average annual discharge while other investigators have used dominant (Inglis, 1940) and bankfull discharges (Nixon, 1948). Inglis (1940) originally defined dominant discharge as:

> the discharge which controls the meander length and breadth. It appears to be slightly in excess of bankfull stage.

However, it is somewhat uncertain whether the discharge that forms the channel is similar to the discharge that controls the meander wave-length. Recently, Benson and Thomas (1966) have defined the term as:

> the discharge that over a long period transports the most sediment.

They, however, defined sediment as suspended load and assumed that the total load is proportional to the suspended load. The importance of the suspended load as compared to the bed-load in the formation of a channel has been discussed by several investigators (Blench 1969, Chien 1955).

At the moment there is no widely accepted discharge that is used by all investigators in analyzing natural rivers or channels, however, in designing channels the design flow can usually be taken as the bankfull flow. An allowance for freeboard is then added to the corresponding bankfull stage to ensure against overtopping. The hydraulic geometry

of coarse-bed rivers will therefore be correlated to bankfull discharge in the following section.

6.3 WIDTH AND DEPTH RELATIONSHIPS

Engineers in India have long been aware that the width, depth and slope of stable irrigation canals are strongly correlated with the bankfull discharge and the type of sediment through which the canal flows. Although many equations have been developed by many individuals the most commonly used are those of Lacey (1929) and Blench (1969). These equations, pertaining to sand-bed canals, are presented in TABLE 10.

Regime equations for coarse-bed channels have been developed by Kellerhals (1967) and Maddock (1969) and are presented in TABLE 11. Kellerhals (1967) equations were developed from his own field data as well as data collected by Lane and Carlson (1953). Maddock (1969) has developed two sets of equations based on the San Luis Canal data of Lane and Carlson (1953); the first set of equations, group (a) in TABLE 11, represent steep canals having discharges larger than 500 cfs while the second set are for discharges less than 500 cfs.

The previously quoted data from Kellerhals (1967) and Lane and Carlson (1953) have been combined with data from the North Saskatchewan River as well as other Alberta rivers to obtain the last set of equations in TABLE 11.

TABLE 10

REGIME EQUATIONS - SAND-BED CANALS

Author	Width ft.	Depth ft.	Slope
Lacey	$P = 2.67Q_b^{1/2}$	$R = \frac{0.472Q_b}{f}^{1/3}$	$S = \frac{0.000547f^{5/3}}{Q_b^{1/6}}$
Blench	$b_w = \left(\frac{Fb}{Fs}\right)^{1/2} Q_b^{1/2}$	$d_* = \left(\frac{Fs}{F_b^2}\right)^{1/3} Q_b^{1/3}$	$S = \frac{0.0005 F_b^{5/6} Fs^{1/12}}{Q_b^{1/6}}$
Nixon	$b_w = 1.65\ Q_b^{1/2}$	$d_* = 0.545\ Q_b^{1/3}$	-

where:

P = wetted perimeter

b_w = width

Q_b = bankfull discharge

R = hydraulic radius

d_* = mean flow depth

f = Lacey's silt factor

F_b = Blench's bed factor

F_s = Blench's side factor

S = Slope

TABLE 11

REGIME EQUATIONS - COARSE-BED CHANNELS

Author	Width ft.	Depth ft.	Slope
Kellerhals	$b_w = 1.8Q_d^{0.50}$	$d_* = \frac{0.166Q_d^{0.40}}{k_s^{0.12}}$	$S = \frac{0.12k_s^{0.92}}{Q_d^{0.4}}$
Maddock (a)	$b_w = 3.8Q_b^{0.382}$	$d_* = \frac{0.041Q_b^{0.549}}{D_{50}^{0.33}}$	$S = \frac{0.51\ D_{50}^{1.17}}{Q_b^{0.48}}$
(b)	$b_w = 3.8Q_b^{0.382}$	$d_* = \frac{0.046Q_b^{0.549}}{D_{50}^{0.33}}$	$S = \frac{0.27\ D_{50}^{1.17}}{Q_b^{0.48}}$
This thesis	$b_w = 2.0Q_b^{0.50}$	$d_* = 0.32Q_b^{0.32}$	$S = \frac{0.051D_{50}^{0.90}}{Q_b^{0.25}}$

where:
Q_d = dominant discharge
b_w = water surface width
k_s = 90% size from grid sample
b_{50} = 50% size from grid sample
Q_b = bankfull discharge
S = slope
d_* = mean flow depth

The width and depth equations were obtained from FIGURE 92 (data tabulated in TABLE A-11, APPENDIX 8). The data, although sparse, do not scatter excessively. Kellerhals' data have been modified slightly, the discharges for the Cariboo River at Cariboo Lake and the Chilko River at Chilko Lake were reduced from dominant to bankfull.

An attempt to relate the slope to bankfull discharge met with little success (FIGURE 93) which is not surprising since many coarse-bed rivers have their slopes controlled by bed-rock outcrops. The slope equation presented in TABLE 11:

$$S = \frac{0.051 \; D_{50}^{\;0.90}}{Q_b^{\;0.25}} \qquad (6.3)$$

was arrived at by combining the flow resistance formula derived for immobile channels (equation 5.9) with the width and depth equations from FIGURE 93.

6.4 COMPARISON OF REGIME EQUATIONS FOR COARSE-BED CHANNELS

It may now be in order to compare the several sets of regime equations for coarse-bed channels. Consider a hypothetical river having a relatively straight alignment with the following characteristics:

$$Qb = Qd = 20{,}000 \text{ cfs}$$

$$D_{50} = 3.0 \text{ inches}$$

$$D_{90} = k_s = 5.5 \text{ inches}$$

The widths, depths, and slopes for this river as computed from the regime relations are as follows:

Author	Width (bw) ft.	Depth (d_*) ft.	Slope
Kellerhals	255	9.5	0.00120
Maddock	168	16.8	0.00046
This thesis	282	7.6	0.00120

The numerical values for widths, depths, and slopes are almost identical for Kellerhals' equations and those developed in this thesis. However, the computed cross-section as based on Maddock's equations is relatively narrow and deep while its slope is about half that computed from the other regime equations. The data used by Maddock to develop his equations are, however, not as extensive as those used in this thesis.

6.5 NON-DIMENSIONAL RATING CURVE

Relatively few investigators record bankfull flow as pertinent hydraulic data. However, it appears that this information is most useful in comparing river behaviour.

Leopold, Wolman and Miller (1964) noted that rating curves of various stations are very similar in form and could be plotted in non-dimensional form by plotting the ratios:

$$\frac{\text{mean depth, } d_*}{\text{bankfull depth, } d_b} \quad \text{versus} \quad \frac{\text{discharge, } Q}{\text{bankfull discharge, } Qb}$$

These non-dimensional relationships were plotted for various coarse-bed rivers in Alberta (FIGURE 94) with reasonable sucess. The values used in the graph are tabulated in TABLE 12.

This graph will be valuable in the estimation of bankfull discharge for coarse-bed rivers in Alberta, and in assessing stable cross-section dimensions. In order to assess the bankfull discharge it is necessary to know the mean depth at a definite discharge as well as the mean depth at bankfull flow. This would require stream gauging as well as the survey of several cross-sections.

With the bankfull discharge estimated the stable width of the river can be assessed from FIGURE 92. This value could be compared to the measured width from field surveys to establish whether the river is in a stable state. This plot can also be used to develop a rating curve for a river provided that the bankfull discharge and depth are known.

It is interesting to note that the recurrence interval for bankfull flows in Alberta Rivers varies from 5 years to 15 years which does not correspond to the recurrence interval of 1.5 years quoted by Leopold, Wolman and Miller (1964) for rivers in Eastern United States. The higher recurrence interval is also characteristic of rivers in British Columbia as reported by Kellerhals (1963).

TABLE 12

NON-DIMENSIONAL RATING CURVE DATA

COARSE-BED CHANNELS

River	Q_b cfs	Q cfs	d_* ft.	$Q/_{Qb}$	d_*/d_b
North Sask.	80,000		15.0	-	-
at Drayton	(15yrs.	7,000	4.3	0.09	0.29
Valley	return	10,000	5.2	0.13	0.35
(Galay, 1967a)	period)	20,000	7.4	0.25	0.49
○		30,000	9.1	0.38	0.61
		40,000	10.4	0.50	0.70
		60,000	13.2	0.75	0.88
		80,000	15.0	1.00	1.00
Red Deer R.					
at Red Deer	30,000		11.7	-	-
(Qureshi,	(10 yr.	1,200	2.1	0.04	0.18
1962 and	return	3,000	3.5	0.10	0.30
Neill, et.al	period)	5,000	4.5	0.17	0.39
1964)		8,000	5.7	0.27	0.49
△		10,000	6.4	0.33	0.55
		20,000	8.9	0.67	0.76
		30,000	11.7	1.00	1.00
Highwood R.					
(Alta.W.Res.)	6,000		5.3	-	-
⧄	(4 yr.	1,000	2.0	0.17	0.38
	return	2,000	3.0	0.33	0.57
	period)	3,000	3.7	0.50	0.70
		4,000	4.3	0.67	0.81
		5,000	4.8	0.83	0.91
		6,000	5.3	1.00	1.00

6.6 CONCLUSIONS

Regime equations developed by Kellerhals, Maddock and the writer have been compared and those developed by Kellerhals were found to compare closely with those developed in this thesis. The equations developed in this thesis, for coarse-bed rivers are:

$$b_w = 2.0\ Q_b^{1/2} \tag{6.1}$$

$$d_* = 0.32\ Q_b^{0.32} \tag{6.2}$$

and

$$S = \frac{0.051\ D_{50}^{0.90}}{Q_b^{0.25}} \tag{6.3}$$

These equations are based on bankfull discharge and the 50% bed-material size as determined from a grid sample analyzed by number. They are tentative as they are based on rather sparse data and on rivers that may not have been completely stable.

A chart having non-dimensional parameters d_*/d_b versus Q/Q_b has been developed from which a rating curve can be deduced. It was noted that the recurrence interval for bankfull flows for Canadian rivers flowing from the Rocky Mountains ranged from 3 to 15 years which is somewhat higher than the recurrence interval for rivers in Eastern United States.

6.7 RECOMMENDATIONS

The collection and analysis of more data from stable coarse-bed irrigation canals would be desirable in order that regime-type equations can be derived with more confidence.

The hydraulic geometry of coarse-bed channels at one location, as well as along a channel, should be studied in more detail. It is possible to develop a rating curve for a channel, provided that the bankfull discharge and depth are known, from the non-dimensional rating curve. However, if the cross-section characteristics of coarse-bed channels were to be known in more detail dimensionless relationships between the flow characteristics and channel characteristics may be developed.

CHAPTER VII

SCOUR AT BENDS OF COARSE-BED RIVERS

7.1 INTRODUCTION

In locating highway and pipeline crossings it is important to have a reasonable estimate of the maximum depth of scour which usually occurs at bends and constrictions.

This chapter will present depths of scour at bends of coarse-bed rivers in Alberta and relate them to the hydraulic characteristics of the river. A discussion of scour and fill at river bends, which appears to be absent in coarse-bed rivers, is also presented.

7.2 SCOUR AT BENDS OF ALLUVIAL CHANNELS

Alluvial channels have been defined as:

> open channels formed in granular, noncohesive material that has been transported to its present site by flow in the channel

(Task Force on Bed Forms in Alluvial Channels, 1966). With this definition it would be in order to describe a coarse-bed river having sand, gravel, cobbles, and boulders on the bottom as an alluvial river. The phenomenon of scour and fill at bends of sand rivers is now generally familiar to river engineers and was well documented by Lane and Borland (1954) who stated that:

> at high-water stages, therefore, the bends usually scour out and deposition occurs at the crossings; whereas during low-water stages, the crossings scour out and the bends fill .

This fact has been supported by echo soundings during the passage of a flood on the Beaver River in Alberta (Neill, 1965) (see FIGURE 95) and by soil borings after a flood on the Umfolozi River in South Africa (Zwamborn, 1966) (see FIGURE 96). The scour and fill process on the Umfolozi River was also accurately reproduced in a hydraulic model with coal as the bed-material. It should be noted that in both of these reported cases the bed-material was fine or medium sand.

During the summer months of 1965 and 1966 the writer was engaged on river regime surveys of coarse-bed rivers in Alberta. One of the rivers investigated in some detail was the North Saskatchewan at Drayton Valley (Galay, 1967a) (see FIGURE 1 for location) which experienced fairly high flows in 1965 and 1966 (see FIGURE 97). Successive cross-section soundings at various bends before, during and after the peak flow revealed several interesting facts (see FIGURES 98 and 99). Most of the bends did not experience any lowering of their bed levels during peak flow. There was some scour at cross-sections 100 and 102 but this appears to be due to a shift of the channel in conjunction with a

reduction of the bend radius. FIGURE 100 illustrates the change in the channel pattern which took place during the passage of the 1965 peak flow. Soundings at the bridge (FIGURE 101) during low flow in 1967 indicate that the scour hole has not filled in.

Some of the other cross-sections also show that the scour holes did not fill in as the flow receded.

Subsequent to this river survey at Drayton Valley a continuous longitudinal sounding along the rivers' thalweg from Drayton Valley to Edmonton was carried out. This sounding was conducted when the river was relatively low and indicated deep scour holes at river bends. Again it appeared that there was little filling of the scour holes during recession of peak flows (FIGURE 102).

Longitudinal soundings along the midstream of a sand-bed river (Neill, 1965) indicate that the depth at bends was approximately equal to twice the mean depth of flow, even for low flow conditions (FIGURE 103). However, for the North Saskatchewan River the depth at bends was some three to five times the mean flow depth as shown by FIGURE 102.

In the summers of 1967 and 1968 a river depth profiling program was carried out on several Alberta rivers. The purpose of this program was (Hollingshead and Schultz,

1968):

> to obtain continuous longitudinal depth profiles along major rivers, and conduct detailed depth surveys at bridges, sharp bends and other locations where scour holes occur .

The rivers surveyed in this program were the North Saskatchewan downstream of Edmonton, the Oldman and the Athabasca (see FIGURE 1 for location).

The survey methods used were:

1. Longitudinal Soundings

   Depth profiles were obtained by sounding along the river's thalweg with fix points being located from topographic maps or airphoto mosaics and the transverse position by a range finder.

2. Cross-Sections

   Soundings of the river-bed and ground surveys of the banks were carried out at right angles to the thalweg. At each cross-section information as to the bank and bed material, vegetation, erosion, etc. was recorded.

3. Scour Holes

   At bends showing deep scour holes additional cross-sections and longitudinal soundings were taken in order to determine the extent of the scour.

All these coarse-bed rivers exhibited relatively deep scour holes at their bends as shown by the soundings taken on the Oldman River (FIGURE 104). An important question now arises - why don't the scour holes at bends of coarse-bed rivers fill in during the recession of peak flows?

The answer to this may lie in the phenomenon known as "bed paving or armouring". This phenomenon has received attention in rivers directly below dams. The bed-load in this case is trapped in the reservoir resulting in a deficiency in bed-load downstream of the dam. The river then erodes its bed and/or banks in order to pick up more sediment. This erosion of the bed continues until the bed is covered with stones that are too large to move. This phenomenon will occur in rivers having sand beds with small quantities of gravel as noted by Livesey (1963) reporting on the Missouri River. The rivers investigated in Alberta, however, have bed-material which is predominantly larger than 1/2 inch in size resulting in an armoured bed occurring more frequently. All coarse-bed rivers display a definite shingled or armoured effect during low-water stages (PHOTOGRAPH 11). This armouring must have taken place at some stage of flow higher than the reference low stage and may account for the fact that the scour holes do not fill in with receding flows. Let us now examine what may happen to

the bed of a river during the passage of a peak flow (see FIGURE 105).

As a flood wave passes a definite river cross-section the bed will remain immobile until the mean velocity is large enough to cause a reasonable number of the smaller particles to start moving (case b, FIGURE 105). As the discharge and velocity increase the larger particles will be moved in a rolling or saltating manner (Durand, 1951 and Thompson, 1965) with the finer particles being carried off in a rush (case c, FIGURE 105). As the flow recedes the larger particles will segregate from the finer ones and drop to the bed to form a paved or semi-paved bed. Harrison, (1950) reported that:

> a complete layer of non-moving particles is not necessary in preventing scour ... only 50 percent completeness is probably required .

The finer bed-material in motion will continue to move with the flow, but there will not be any further supply of fine particles from the river-bed due to the paving; the flow would be capable of carrying more material along the bed than would be available from the bed. Therefore, the particles still in motion would be comparatively small and easy to transport resulting in little or no deposition along the river's thalweg. Strong helicoidal currents would be present at the bends of rivers which would sweep out any fine gravel or sand that may be transported into the scour holes. The

strength of these helicoidal currents can be attested to by the fact that several 120 lb. concrete "blocks" were deposited on the top of a high point bar at the inside of a bend on the North Saskatchewan River (PHOTOGRAPH 9).

The hypothesis that scour holes at bends do not fill in during low stages makes it possible for river-bend surveys to be carried out at low stages on coarse-bed rivers. It should be possible to correlate the scour data to pertinent hydraulic variables.

7.3 TYPES OF RIVER BENDS

Prior to setting out the variables that would influence the depth of scour it may be in order to consider the types of river bends that exist in nature.

From field and airphoto investigations it is apparent that there are at least two significantly different types of bends:

1. Free bends
2. Forced bends.

A free bend is formed by a river flowing on a flood plain which is free to erode its banks and to migrate laterally or longitudinally (FIGURE 106). A forced bend, however, usually impinges on a valley wall resulting in an abrupt change in flow direction. The radius of curvature is usually larger for free bends, but the depth of scour is usually smaller because the scouring action is directed at the bank as well

as the bed of the stream.

Rzhanitsyn, (1960) has also classified river bends into several distinct categories namely free bends, limited bends and forced bends; this was, however, for sand-bed channels. This discussion will consider only the two categories originally mentioned.

7.4 DIMENSIONAL ANALYSIS

Dimensional analysis of the depth of scour in relation to the channel properties and the pertinent hydraulic variables will now be carried out. This analysis will pertain to a forced bend flowing at bankfull stage (see FIGURE 107).

It would appear that the depth of scour d_s, which is measured from the water surface, is dependent upon the following variables:

Fluid properties

ρ = density

μ = dynamic viscosity

Flow properties

Q_b = bankfull discharge

S = water surface slope

d_* = mean flow depth upstream of bend at bankfull stage

Channel properties

b_w = water-surface width at bend

r = radius of curvature

θ = internal angle(in radians)

Bed-material properties

D = typical particle size

ρ_s = density

σ_b = gradation of bed-material

Bank material properties

M = type of bank material

and

g = acceleration of gravity

In equation form:

$$d_s = f_1 (\rho, \mu, Q_b, S, d_*, b_w, r, \theta, D, \rho_s, \sigma_b, M, g) \tag{7.1}$$

The bankfull discharge Q_b can be replaced by Vm through the continuity equation. Using the Buckingham π theorem with ρ, Vm and b_w as repeating variables results in:

$$\frac{d_s}{b_w} = f_1 \left(\frac{\rho V m b w}{\mu}, \frac{Vm}{\sqrt{g b_w}}, S, \frac{d_*}{b_w}, \frac{r}{b_w}, \theta, \frac{D}{b_w}, \frac{\rho_s}{\rho}, \sigma, M\right) \tag{7.2}$$

Some of the terms can be modified by combination with the d_*/b_w term:

$$\frac{d_s}{b_w} = f_2 \left(\frac{\rho V m d_*}{\mu}, \frac{Vm}{\sqrt{g d_*}}, S, \frac{d_*}{b_w}, \frac{r}{b_w}, \theta, \frac{D}{d_*}, \frac{\rho s}{\rho}, \sigma, M\right) \tag{7.3}$$

The terms in this equation can be reduced by considering the following assumptions:

(1) Fully rough turbulent flow conditions will prevail making the Reynolds number of secondary importance.

(2) The bed-material is similar in size, gradation and density from one river to the next making it possible to discard $\rho s/\rho$ and σ.

(3) The bank material for fixed bends is assumed to be of resistant bedrock varying little from one bend to another; the term M is deleted.

These deletions result in:

$$\frac{ds}{b_w} = f_3\left(\frac{Vm}{\sqrt{gd}}, S, \frac{d_*}{b_w}, \frac{r}{b_w}, \theta, \frac{D}{d_*}\right) \qquad (7.4)$$

At this stage it is difficult to reduce the equation to fewer dimensionless terms.

The significant variables affecting scour would be the radius of curvature r and the internal angle θ; the depth of scour would definitely be larger for a river with a severe bend as compared to a gradually winding river (see FIGURE 108). The importance of the Froude number $\frac{Vm}{\sqrt{gd}}$, the slope S, and the ratios d_*/b_w and D/d_* are difficult to assess; insufficient data regarding bankfull flow in coarse-bed rivers makes it difficult to graphically or statistically check their influence. In the following analysis it will be assumed that the effect of radius of curvature r and the internal angle θ and the width b_w are dominant resulting in:

$$\frac{d_*}{b_w} = f_4 \left(\frac{r}{b_w}, \theta\right) \qquad (7.5)$$

7.5 ANALYSIS OF SCOUR DATA FOR COARSE-BED RIVERS IN ALBERTA

The previously mentioned dimensionless variables are tabulated in TABLE 13 and have been obtained from field surveys of Alberta rivers.

A log-log plot of $\frac{d_s}{b_w}$ vs $\frac{r}{b_w}$ with θ as the third variable was found to have a large degree of scatter as shown in FIGURE 109. It is somewhat difficult to establish lines having distinct θ values, however, lines having θ equal to 1.5 and 2.0 are tentatively sketched on.

A subsequent plot of $\frac{d_s}{b_w}$ vs θ for $\frac{r}{b_w}$ in the range 2.5 to 3.5 (FIGURE 110) indicates that $\frac{d_s}{b_w}$ is related linearly to the internal angle of the river bend. Since θ effects the depth of scour in a linear manner it was arbitrarily combined with the term r/b_w and a plot of d_s/b_w versus $\frac{r}{b_w \theta}$ for fixed bends is shown in FIGURE 111. A "design line" was fitted through the top of the plotted points as engineers would be primarily interested in the maximum possible scour at a river bend. The relationship derived from this plot is:

$$d_s/b_w = 0.14 \left(\frac{r}{b_w \theta}\right)^{-0.48} \qquad (7.6)$$

The data for free bends was plotted in the same manner (FIGURE 112) yielding another design line. A comparison of the two "design lines" shows the depth of scour for

TABLE 13

SCOUR AT RIVER BENDS

HYDRAULIC PROPERTIES OF BENDS AT BANKFULL STAGE

RIVER	LOCATION (x-sec.)	d_s (ft.)	b_w (ft.)	r (ft.)	θ	θ (radians)	d_s/b_w	r/b_w	$r/b_w\theta$
				FORCED BENDS					
Oldman	51	13.9	430	1,800	60°	1.05	0.032	4.2	3.7
	57	12.3	470	7,500	30	0.52	0.026	16.0	31.0
	23	38.2	305	1,050	150	2.62	0.125	3.5	1.3
	64	15.4	350	2,500	80	1.40	0.044	7.1	5.1
	72	26.6	360	1,000	80	1.40	0.074	2.8	2.0
	73	25.6	410	1,300	135	2.36	0.062	3.2	1.4
	74	25.1	395	1,700	150	2.62	0.063	4.3	1.6
	85	18.0	550	3,000	75	1.31	0.033	5.5	4.2
	87	22.0	480	1,600	65	1.13	0.046	3.3	2.9
	29	19.3	220	650	125	2.18	0.088	3.0	1.4
North Sask. (Drayton Valley)	67	23.0	550	1,400	60°	1.05	0.042	2.6	2.5
	78	24.0	560	1,000	60	1.05	0.043	1.8	1.7
	88	22.0	650	2,000	35	0.61	0.034	3.1	5.1
	100	28.0	400	800	90	1.57	0.070	2.0	1.3
	102	31.0	650	1,600	45	0.79	0.047	2.9	3.7
	110	21.0	550	1,800	35	0.61	0.038	3.3	5.4
	115	22.0	430	2,500	55	0.96	0.051	5.8	6.0
North Sask. (near Edmonton)	14	42.0	600	850	90°	1.57	0.070	1.4	0.9
	21	30.0	650	3,000	90	1.57	0.046	4.6	2.9
	33	38.0	530	1,540	75	1.31	0.072	2.9	2.2
	39	43.0	490	690	85	1.49	0.088	1.4	1.0
	43	30.0	700	4,830	45	0.79	0.043	6.9	8.8
	40	46.0	450	790	90	1.57	0.102	1.8	1.1
Athabasca	147	21.0	560	2,100	50°	0.87	0.038	3.7	4.3
	166	28.0	410	970	90	1.57	0.068	2.4	1.5
	172	33.0	480	3,700	50	0.87	0.069	7.6	8.7
	175	49.0	420	800	120	2.10	0.117	1.9	0.9
	178	34.0	400	850	80	1.40	0.085	2.1	1.5
	180	42.0	500	2,100	90	1.57	0.084	4.1	2.6
	185	40.0	440	800	90	1.57	0.091	1.8	1.1
	152	26.0	560	2,100	50	0.87	0.046	3.8	4.3
	166A	30.0	330	870	90	1.57	0.091	2.6	1.7
	190	31.0	540	1,200	90	1.57	0.057	2.2	1.4
...	...	...	...	...	...	...	...	...	...
				FREE BENDS					
Oldman	46	12.3	280	3,200	65°	1.13	0.044	11.8	10.5
	79	19.6	410	1,000	75	1.31	0.048	2.5	1.9
	85	18.6	435	3,800	40	0.70	0.043	8.8	12.6
North Sask. (Drayton Valley)	73	18.0	400	2,600	45	0.79	0.045	6.5	8.2
	85	18.0	450	2,900	30	0.52	0.040	6.4	12.3
	96	19.0	450	1,200	90	1.57	0.042	2.7	1.7
North Sask. (near Edmonton)	54	38.0	650	1,320	60	1.05	0.058	2.0	1.9
	102	40.0	650	1,590	55	0.96	0.062	2.5	4.2
Athabasca	146	21.0	600	4,800	30	0.52	0.035	8.0	15.4
	194	32.0	620	3,500	50	0.87	0.052	5.6	6.4
...	...	...	...	...	...	...	...	...	...

the same value of $\frac{r}{b_w \theta}$ will be larger for a forced bend (see FIGURE 113). This confirms the previous statement that a free bend will have caving banks resulting in shallower scour holes.

The fact that the "design plots" (FIGURES 111 and 112 possess a fair degree of scatter may be attributed to the neglect of several variables as discussed previously as well as the following factors:

(1) The assessment of the bankfull stage for the Oldman and the Athabasca Rivers was tentative. Plotting of all the available cross-sections on the slope profiles would result in more accurately established stages.

(2) The radius of curvature r and the internal angle θ were obtained from aerial photos for some rivers and from topographic maps for parts of the Oldman River.

(3) The survey dates of the rivers investigated did not coincide with the dates on the air-photos. In some instances it was evident that the surveyed channel had shifted somewhat since the photographs were taken.

(4) The arc of curvature was taken as being parallel and midway between the river banks which in many instances would not coincide

with the thalweg of the river. This is clearly indicated in FIGURE 114 which shows the thalweg as obtained from detailed bed-contours as well the centre-line of the channel.

(5) Some bend cross-sections were of rather complex shapes requiring a rather arbitrary division of the section into an "active" and a "passive" portion (See FIGURE 98). The flow over the shallow portion at the inside of the bend would be retarded much more than the flow in the "active" flow portion and would probably have little influence on the scouring process at the bend.

(6) The scour depth was measured below a bank-full stage, however, some rivers may experience bankfull flow conditions so infrequently that the scour holes at bends have not been fully developed.

(7) The geology from river to river, as well as along the course of an individual river, may vary; some bends may be located at or near subsurface rock outcrops which would limit the full development of the scour hole.

(8) The rivers may have distinctly different channel patterns, such as braided and meandering,

which would influence the development of secondary helicoidal currents.

(9) The bank-material may vary widely; some bends that have been classified as free may have fairly resistant banks.

7.6 PROCEDURE FOR ASSESSING SCOUR AT RIVER BENDS

The plots of d_s/b_w versus $\frac{r}{b_w\theta}$ for forced and free river bends can now be used to assess the maximum scour depth. The use of these plots requires the following data:

(1) Topographic maps and/or airphotos in order to obtain the radius of curvature r and the bend internal angle θ.

(2) Cross-section surveys in order to assess the bankfull width and to obtain a bankfull reference stage below which the depth of scour d_s is to be measured.

The procedure for establishing the bankfull stage has previously been outlined by Neill and Galay, (1967) and proceeds as follows:

(a) Locate and survey a cross-section at the bend in question as well as two cross-sections upstream and two downstream of this bend;

(b) Plot the cross-sections and draw a tentative bankfull stage at each cross-section, without reference to the other sections;

(c) Plot a tentative bankfull stage at each cross-section on the slope profile;

(d) Draw an average bankfull stage line on the slope profile and use this as the reference plane below which the depth of scour is to be measured.

Therefore, with $\frac{r}{b_w \theta}$ computed one obtains a value for $\frac{d_s}{b_w}$ and subsequently d_s. To this depth of scour it may be advisable to add several feet to account for the development of dunes and the fact that the bed of a "gravel" river becomes very active at high flows (Henderson, 1966).

In the training of rivers many free bends will become forced bends with the construction of revetments - the relationship for forced bends should therefore be used in these instances to estimate the depth of scour that one would expect in the future.

7.7 COMPARISON OF DERIVED SCOUR DEPTH RELATIONSHIPS TO REGIME EQUATIONS

The depth of scour for sand-bed rivers can be estimated by using the regime relationships of Lacey (1929) and Blench (1969) and these same relationships will now be assumed applicable to coarse-bed rivers. In order to compare relationships, computations of scour depth were carried out for Prairie Creek near Rocky Mountain House (Alberta Water Resources, 1967). The computed depths of scour, for a free

bend, are as follows:

	LACEY	BLENCH	This Thesis (Figure 112)	Actual Scour
Depth of Scour (ft.)	6	10	11	9

The detailed computation steps are presented in APPENDIX 9. The depth of scour as obtained from Blench's relationships and that derived from FIGURE 112 are very close to the actual scour while the results from Lacey's relationships underestimate the scour depth. However, this comparison needs to be carried out for a large number of bends before more definite conclusions can be made.

7.8 CONCLUSIONS

Longitudinal thalweg soundings on a limited number of coarse-bed rivers in Alberta have revealed that scour holes at bends do not scour and fill during the passing of high flows. This phenomenon is unique to coarse-bed rivers as sand-bed rivers have their scour holes filled in during the recession of high flows. However, more detailed field investigations are necessary in order to conclusively state that scour holes do not scour and fill.

Dimensional analysis was used in an attempt to relate the depth of scour to pertinent channel and flow variables, however, a relationship between the depth of scour and several dimensionless variables was not possible as too many

variables were involved. Treating only the variables of major significance, namely width of river bw, radius of curvature r, and internal angle θ resulted in:

$$\frac{ds}{b_w} = f_4 \left(\frac{r}{b_w}, \theta\right) \tag{7.5}$$

Using data from Alberta Rivers, and classifying bends into forced and free bends resulted in the following tentative design equations:

forced bends:

$$\frac{ds}{b_w} = 0.14 \left(\frac{r}{b_w \theta}\right)^{-0.48} \tag{7.6}$$

and free bends:

$$\frac{ds}{b_w} = 0.11 \left(\frac{r}{b_w \theta}\right)^{-0.38} \tag{7.7}$$

The derived design formula for forced bends was applied to a typical coarse-bed river, Prairie Creek near Rocky Mountain House, and the derived scour depth compared closely to the actual depth as well as to the depth obtained by using Blench's equation for zero flood depth.

7.9 RECOMMENDATIONS

The basic phenomenon of scour at bends of coarse-bed rivers needs far more study. Field surveys using echo sounders, undertaken during the passage of flood flows, are necessary to conclusively state that scour holes do not experience scour and fill. The recording of the modification of bed forms as they pass through scour holes would be of

interest. Surveys should also be conducted at constrictions; the secondary currents at constrictions may differ distinctly from those at bends resulting in a different scour and fill process.

Laboratory research on the transport and segregation of bed-material mixtures under unsteady flow conditions could clarify the bed-paving or sorting process that is so distinct in natural rivers. Also, in hydraulic models of coarse-bed rivers it has been common to use a coarse light-weight material having only one size as the bed material in the model. This uniform size of material may severely restrict the model from reproducing the scour process as it occurs in nature if this process is highly dependent on the sorting of the various sizes. This sorting phenomenon will not occur in the model - one may find that the scour holes will fill in and that the losses would not be reproduced correctly. Model studies of scour at river bends, using various types of materials, would be most informative.

CHAPTER VIII

CONCLUSIONS AND RECOMMENDATIONS

8.1 INTRODUCTION

At the outset of this study the original purpose was to extend the knowledge of coarse bed-material transport by investigating the movement of bed-material in the North Saskatchewan River. However, it soon became apparent that the behaviour of rivers having coarse beds was extremely complex. The study then expanded into an evaluation of the more critical problems confronting engineers dealing with these rivers. The problems considered dealt with bed-material sampling techniques, threshold of motion, bed-material transport, resistance to flow, stable channel design and scour at river bends. After analyzing the available information in each of these topics, and they are by no means completely distinct from each other, conclusions, and recommendations regarding further research were formulated.

8.2 CONCLUSIONS

1. The techniques used in sampling the river-bed are directly related to the purpose of sampling which ususally involves the threshold of motion, the volume of material in transport or the resistance that a coarse bed offers to the flow. Sampling the bed in the following two ways would generally serve the mentioned purposes:

(a) Scoop sample at several depths below the surface, analyzed by weight.

(b) Grid sample by a paced or taped grid, analyzed by number.

2. In order to assess the threshold of motion for coarse-bed rivers having a range of stone sizes field investigations were conducted on the North Saskatchewan River and Wilson Creek. The data obtained were combined with data from other investigators and analyzed in several different ways, namely:

(a) Mean critical velocity related to stone size.

(b) Critical shear stress related to stone size.

(c) Shields mobility number related to particle Reynolds number.

(d) Modified mobility number related to relative depth and

(e) Blench's zero bed factor related to relative depth.

After examining the various relationships the relationship between mean critical velocity and stone size:

$$Vmc = 8.0 \quad D^{1/3}$$

was recommended for assessing when large stones in a mixture will start to move. The velocity in this relationship was the mean velocity in a cross-section, not the mean in a vertical distribution. For this reason it was difficult to directly compare this relationship to similar relationships developed

by other investigators.

Of the various dimensionless relationships investigated that of Blench:

$$F_{bo} = 29 \, (D/d_*)^{1/2}$$

appeared to be reasonable.

3. Existing sediment transport data utilizing coarse materials were analyzed through the use of dimensional analysis. This analysis indicated that there are insufficient data to develop relationships that would be applicable to natural rivers. A subsequent replotting of the data in a simplified form having the unit bed-load discharge versus mean velocity with relative depth as the third variable proved promising. The plotting of data in this form would be more acceptable to practicing engineers.

A comparison of the computed bed-load transport, using several common formulas, to the actual measured transport in the Elbow River indicated that no one formula can be recommended over all the others. Computing the bed-load transport, in the North Saskatchewan River, from shifting gravel bars yielded transport rates that compared favourably with computations based on various formulas.

4. The measurement of the direct protrusion height of stones on a river bed, using a roughness meter, resulted in the following resistance formula being developed for rigid coarse-bed channels:

$$\frac{Vm}{v_*} = 3.0 \left(\frac{d_*}{k}\right)^{0.45}$$

In this formula the protrusion $k = 0.67\ D_{50}$ (by number). The use of direct protrusion height, instead of a computed equivalent grain size, in a flow formula would make it comparable to formulas developed from flume studies with artificial roughness elements.

Engineers may prefer to convert the preceding equation into terms of Manning's roughness coefficient n and this would result in:

$$\frac{n}{d_*^{1/6}} = \frac{0.088}{(d_*/k)^{0.45}}$$

where $k = 0.67\ D_{50}$ (by number).

5. For mobile coarse-bed channels no equations or charts have been developed to assess flow resistance; as in sediment transport, the data are insufficient. Several interesting observations, from flume experiments and field studies, are, however, worth noting. Flume data, for uniform materials, tentatively indicate that the size of material in motion has little influence on resistance. Limited field data suggests that the bed-material, once in motion, forms into dunes and then into a plane bed as the flow intensity increases.

6. For the design of stable channels in coarse bed-material several equations have been compared and those

developed in this thesis are recommended:

$$b_w = 2.0\ Q_b^{0.50}$$

$$d_* = 0.32\ Q_b^{0.32}$$

and

$$S = \frac{0.051\ D_{50}^{0.90}}{Q_b^{\ 0.25}}$$

These equations are based on bankfull discharge and the D_{50} bed-material size as determined from a grid sample analyzed by number. The data used in developing these equations are from rivers in North America and the derived equations may not apply to coarse-bed rivers elsewhere.

7. A chart having non-dimensional parameters $^{d_*}/d_b$ versus $^{Q}/Q_b$ has been developed from which a rating curve for coarse-bed rivers can be deduced. Bankfull discharges can also be estimated if cross-section and flow data at some low flow are available.

8. During river regime surveys, echo soundings at river bends having coarse beds revealed that scour holes, once established, are relatively stable. They do not undergo a scour and fill process as is the case for sand-bed rivers during the passage of a flood.

From an analysis of the dominant variables influencing scour at river bends the following relationships for the

maximum scour depth have been developed:

Fixed bends:

$$\frac{ds}{b_w} = 0.14 \left(\frac{r}{b_w \theta}\right)^{-0.48}$$

Free bends:

$$\frac{ds}{b_w} = 0.11 \left(\frac{r}{b_w \theta}\right)^{-0.38}$$

These equations are tentative as the available data is relatively inconsistent and sparse.

8.3 RECOMMENDATIONS

After treating a number of problems associated with coarse-bed rivers it becomes apparent that adequate field as well as experimental data are lacking. The following recommendations aim at overcoming this deficiency.

8.3(a) Field Investigations

Field investigations should be organized to obtain a wide range of useful data from one river or canal. Several of the problems dealt with, namely the threshold of motion, the transport of sediment, the resistance to flow and the design of stable channels can be investigated simultaneously on one or more rivers. These investigations should be carried out on the following types of channels:

1. Straight irrigation canal or river having a relatively low flow (bankfull discharge up to 5000 cfs).

2. Straight stretch of river having a relatively high bankfull flow (up to 10,000 cfs).

The field program should be organized so that the following information is obtained:

1. Size distribution of surface and subsurface bed-material, as well as bank material.

2. Variation of bed-material sizes with their location in the channel.

3. The hydraulic conditions under which a reasonable portion of the bed is in motion.

4. The amount of bed-material moving along the channel and the portion of the channel bottom that is mobile.

5. The resistance to flow that a rigid bed offers to the flow and the change in this resistance as the bed becomes highly mobile. A continuous record of the bed forms generated would be useful in correlating the bed form dimensions to the corresponding resistance.

6. The values of width, depth, and slope at bankfull flow conditions.

A detailed field investigation on the behaviour of scour holes at bends should also be undertaken. A sounding program to obtain detailed bed contours before, during and

after the passage of flood flows would be necessary.

8.3(b) Laboratory Investigations

At the outset it is stressed that laboratory investigations should deal primarily with coarse bed-material mixtures instead of uniform material.

In studying the threshold of motion the bed-material should initially be sorted by comparatively high flows. The resulting data would therefore be applicable to natural rivers; the placing of material in the bottom of a flume with the use of screeds may result in bed particle orientations that are not representative of field conditions. The threshold of motion state may have to be assessed by hydrophone or by radioactive labelling of particles.

As the intensity of flow increases the bed-material becomes mobile; the continuous measurement of transport rates as well as development and movement of bed forms would yield valuable data on sediment transport and resistance to flow. The investigations should be conducted with mixtures having a median size of at least 20 mm and a relative depth up to 40. These investigations should also be carried out with various materials having a range of densities.

Laboratory investigations dealing with the development of scour holes at river bends having coarse materials may clarify field observations and indicate the more significant variables involved in the scour process. These studies

would require a fairly large model and various types of light-weight materials to simulate bed-material mixtures.

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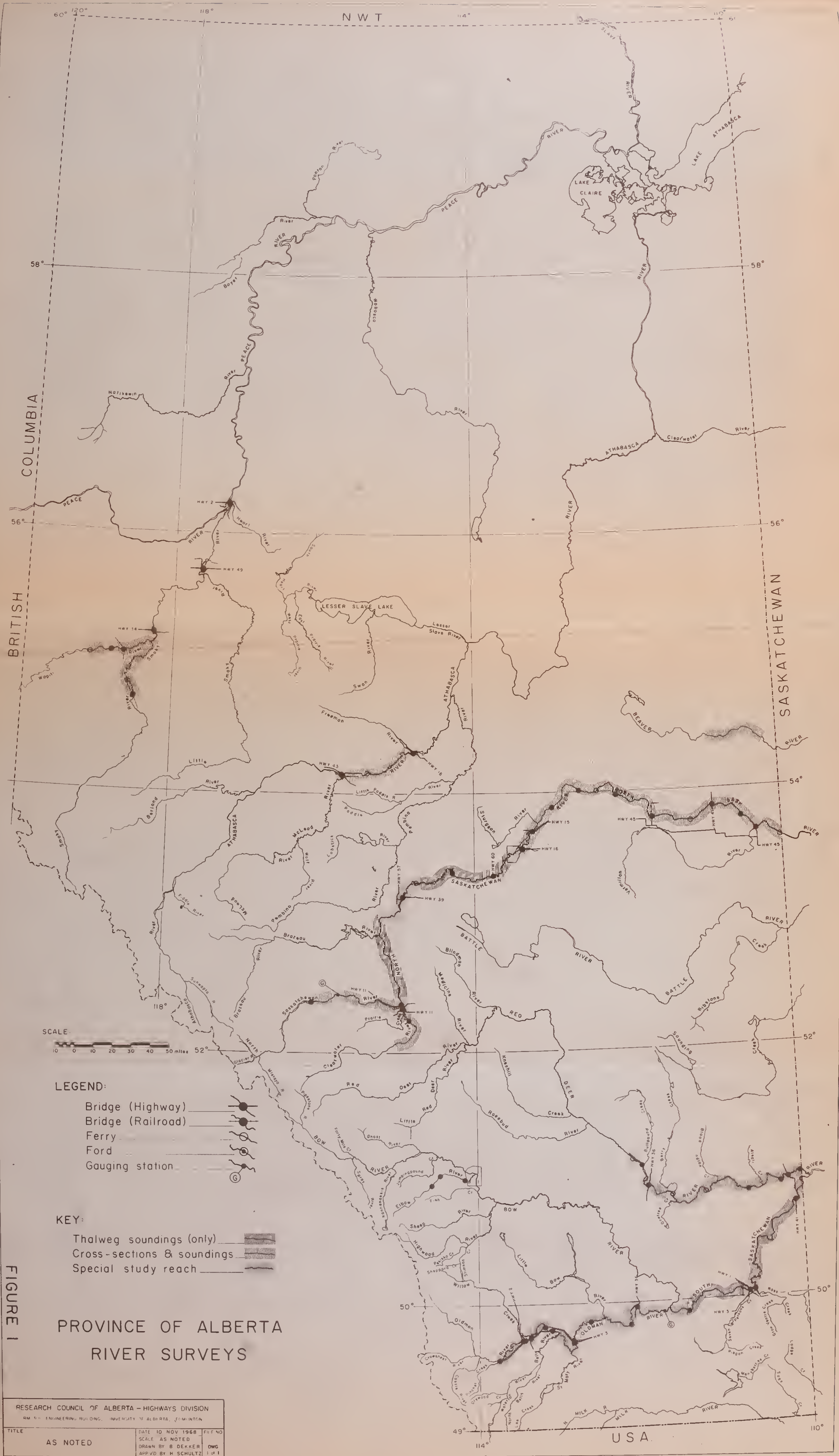
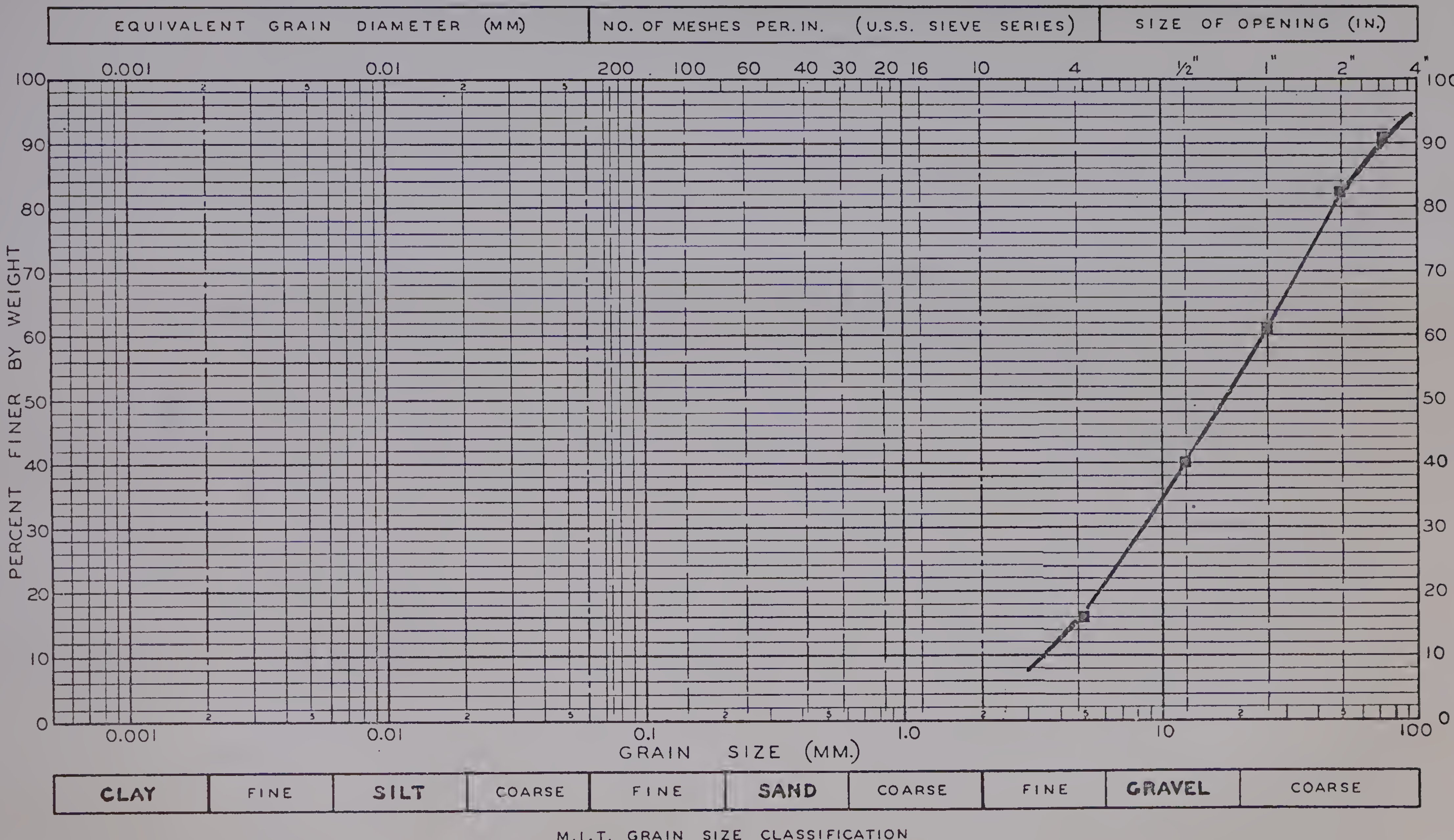
FIGURES



PROVINCE OF ALBERTA
RIVER SURVEYS
NWT
BRITISH COLUMBIA
SASKATCHEWAN
USA
LEGEND:
Bridge (Highway)
Bridge (Railroad)
Ferry
Ford
Gauging station
KEY:
Thalweg soundings (only)
Cross-sections & soundings
Special study reach
SCALE
10 0 10 20 30 40 50 miles
LESSER SLAVE LAKE
LAKE CLAIRE
LAKE ATHABASCA
PEACE RIVER
ATHABASCA RIVER
NORTH SASKATCHEWAN RIVER
SOUTH SASKATCHEWAN RIVER
RED DEER RIVER
BATTLE RIVER
BOW RIVER
OLDMAN RIVER
MILK RIVER
HWY 2
HWY 49
HWY 43
HWY 15
HWY 16
HWY 45
HWY 39
HWY 11
HWY 3
HWY 36
RESEARCH COUNCIL OF ALBERTA - HIGHWAYS DIVISION
TITLE
AS NOTED
DATE 10 NOV 1968
SCALE AS NOTED
DRAWN BY B DEKKER
APP'VD BY H SCHULTZ

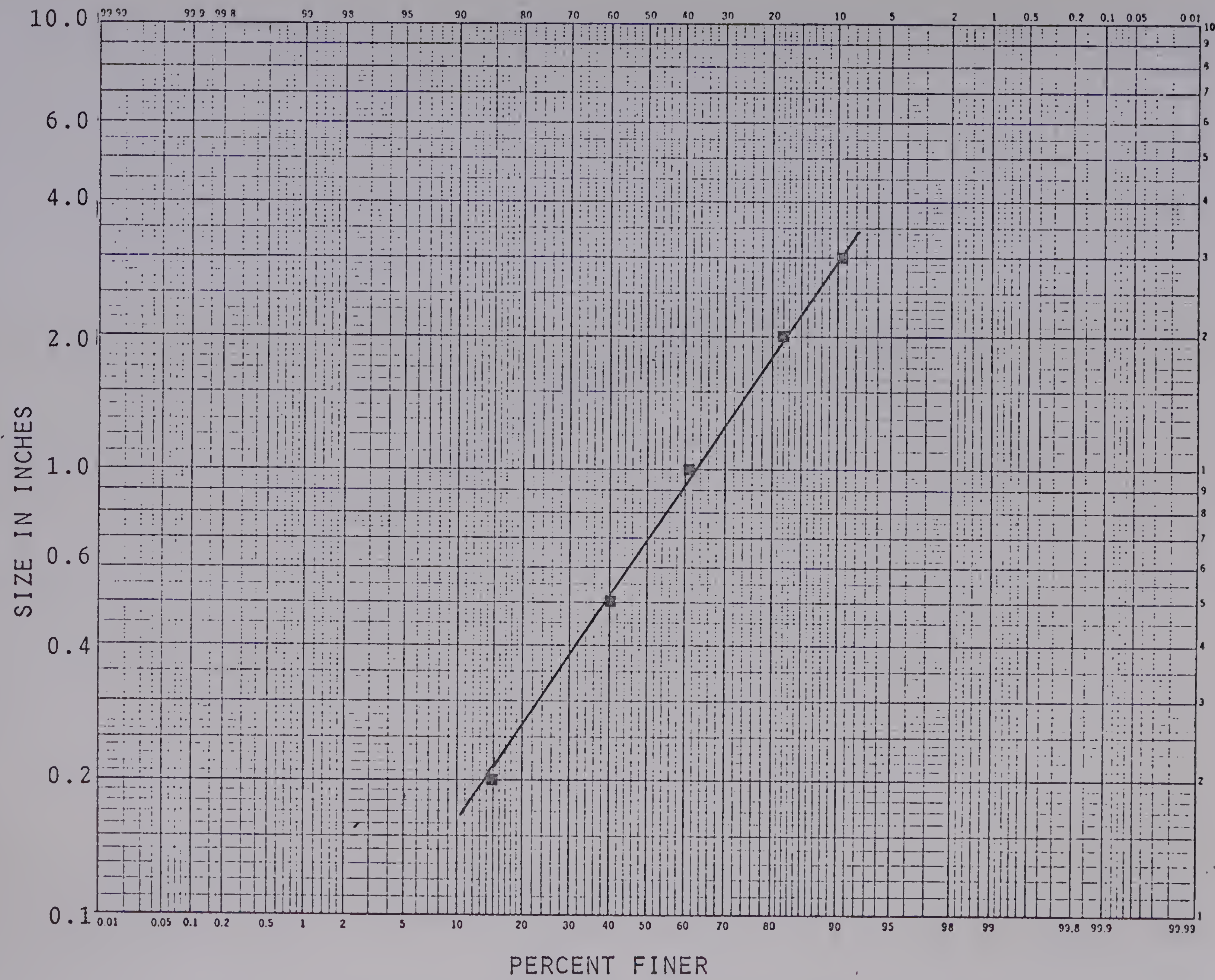
FIGURE I



(Drywood Creek, Sta. 122+00, 6 inches below surface - Van Der Giessen)

STANDARD PLOT FROM A SIEVE ANALYSIS OF A SCOOP SAMPLE

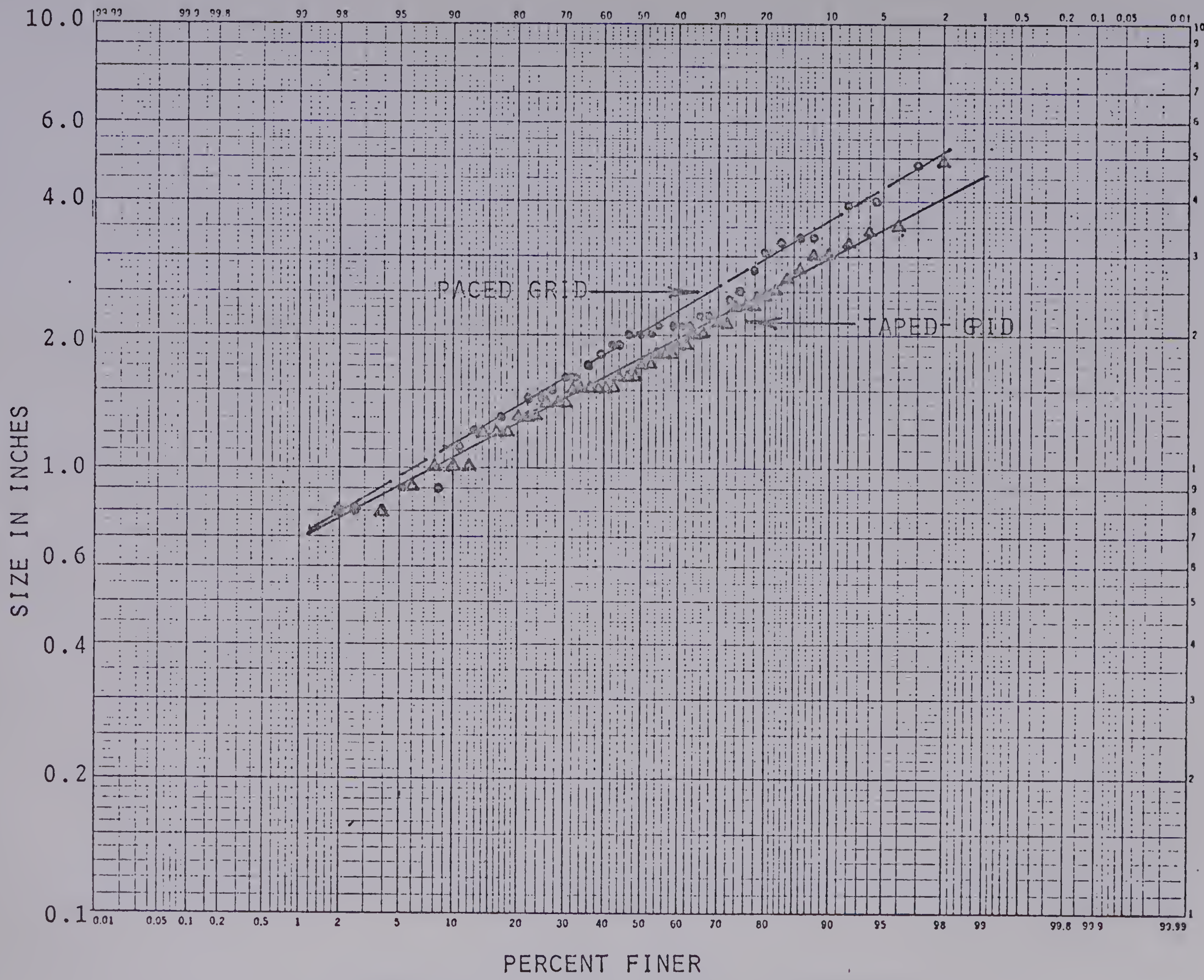
FIGURE 2



(Drywood Creek, Sta. 122+00, 6 inches below surface- Van Der Giessen)

FIGURE 3

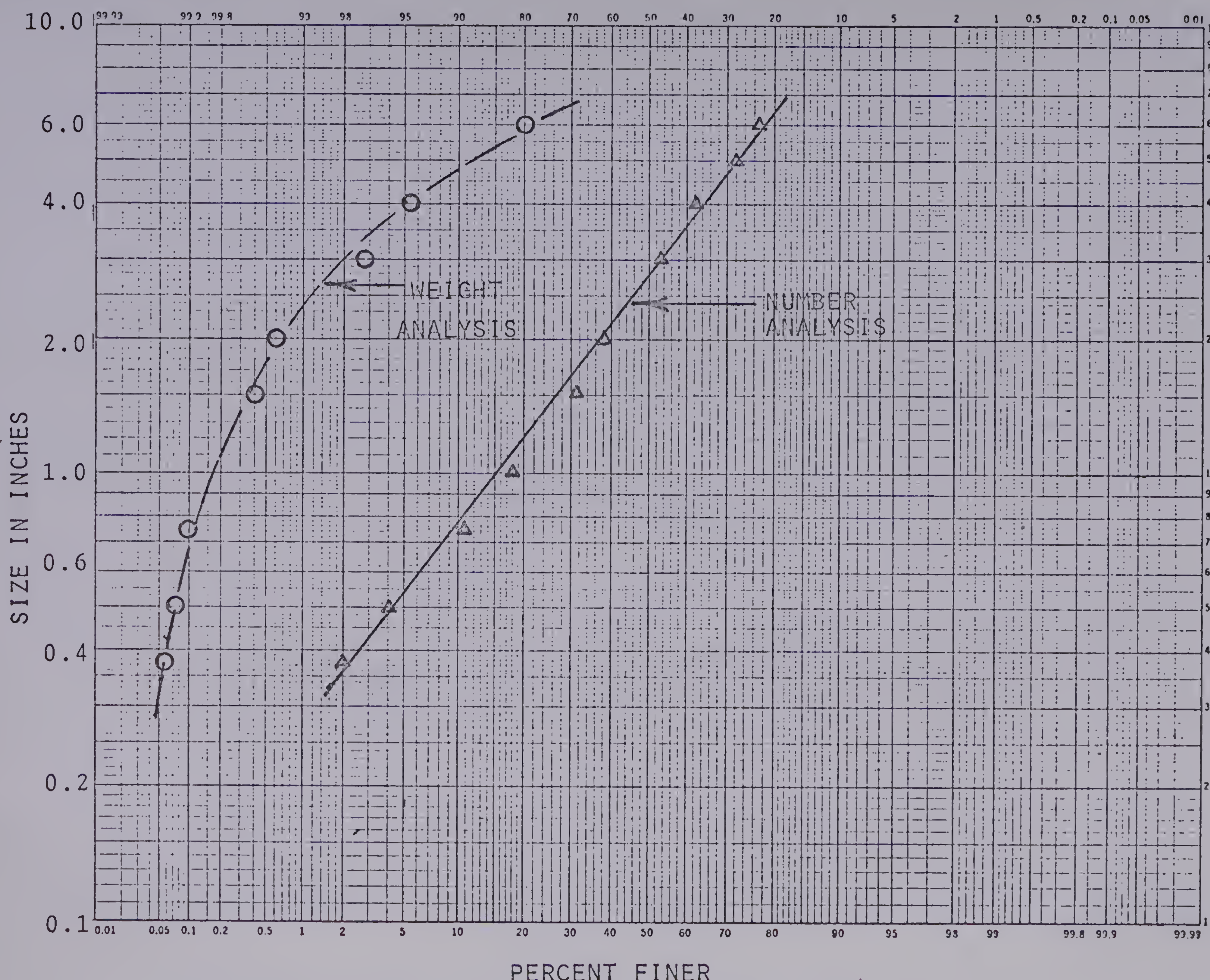
LOG-PROBABILITY PLOT FROM A SIEVE ANALYSIS OF A SCOOP SAMPLE



(North Sask. River- Drayton Valley-x-sec 100)

TAPED-GRID AND PACED-GRID DISTRIBUTION CURVES

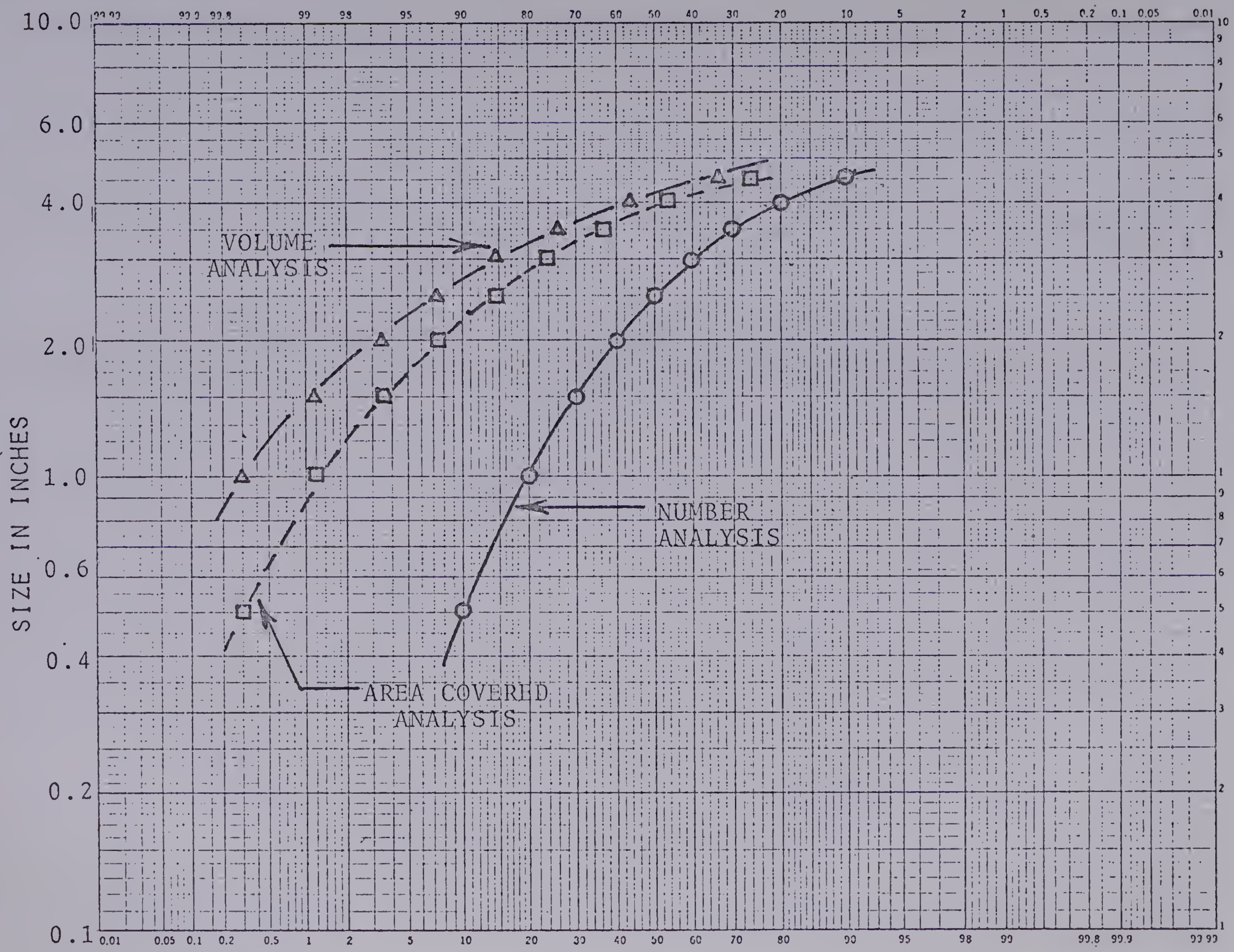
FIGURE 4



(Sheep River- Site A- Van Der Giessen)

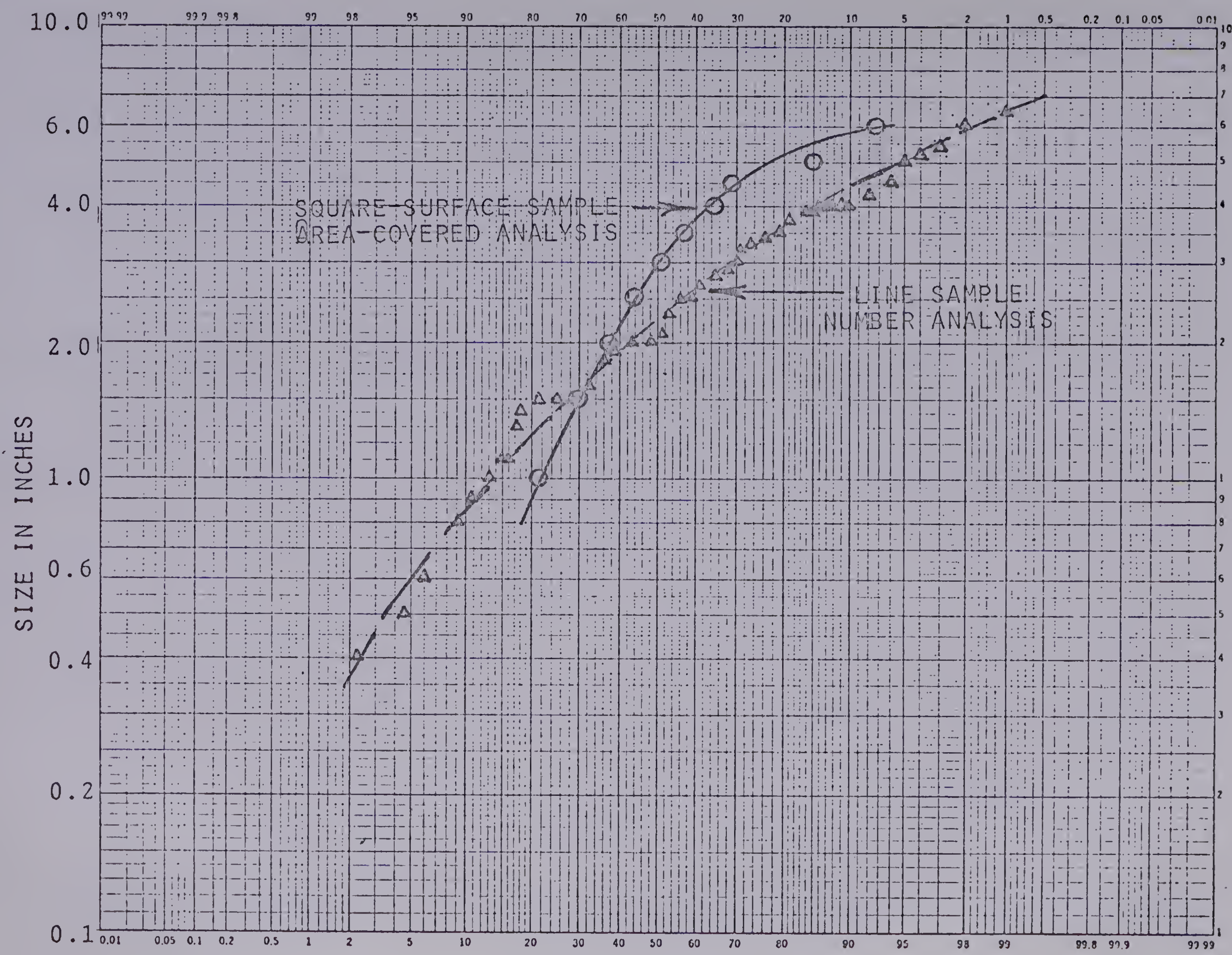
COMPARISON OF A LINE SAMPLE- WEIGHT AND NUMBER ANALYSIS

FIGURE 5



COMPARISON OF HYPOTHETICAL SQUARE-SURFACE SAMPLE
ANALYZED BY NUMBER, VOLUME, AND AREA COVERED

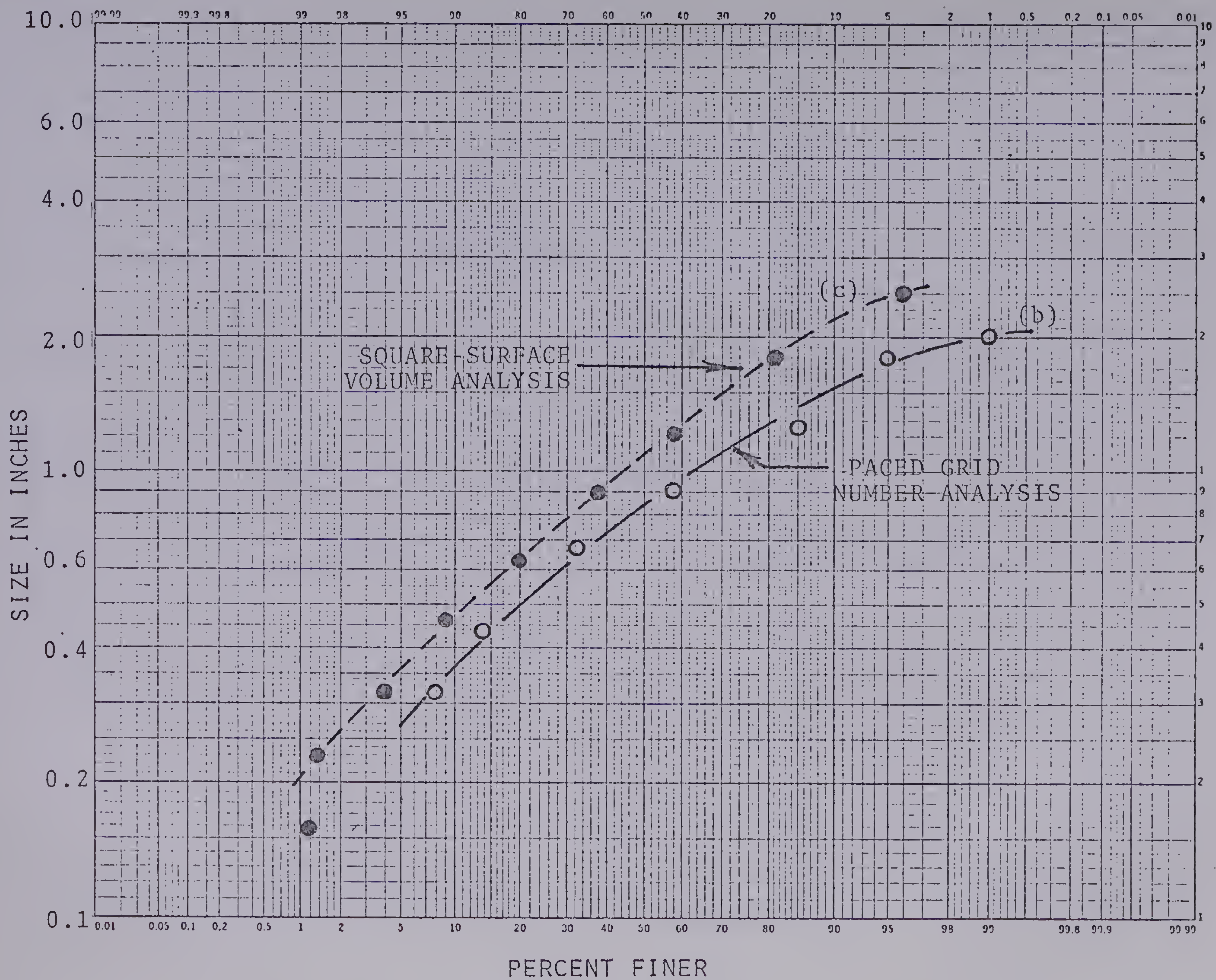
FIGURE 6



(North Sask. River x-sec 87)

COMPARISON OF LINE SAMPLE-NUMBER ANALYSIS WITH SQUARE-SURFACE SAMPLE-AREA COVERED ANALYSIS

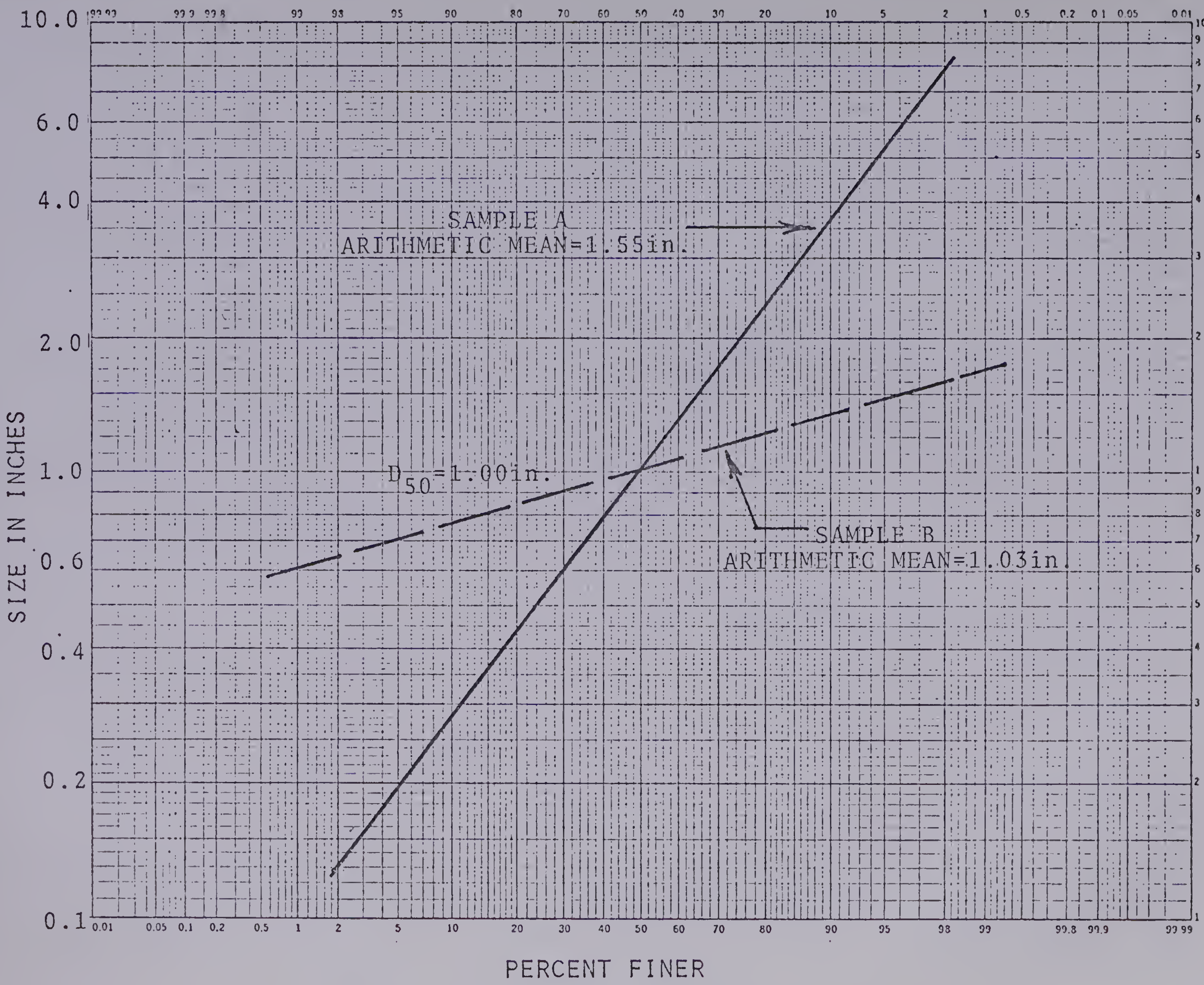
FIGURE 7



(Chowchilla River above Buchanan Damsite- Ritter and Helley 1969)

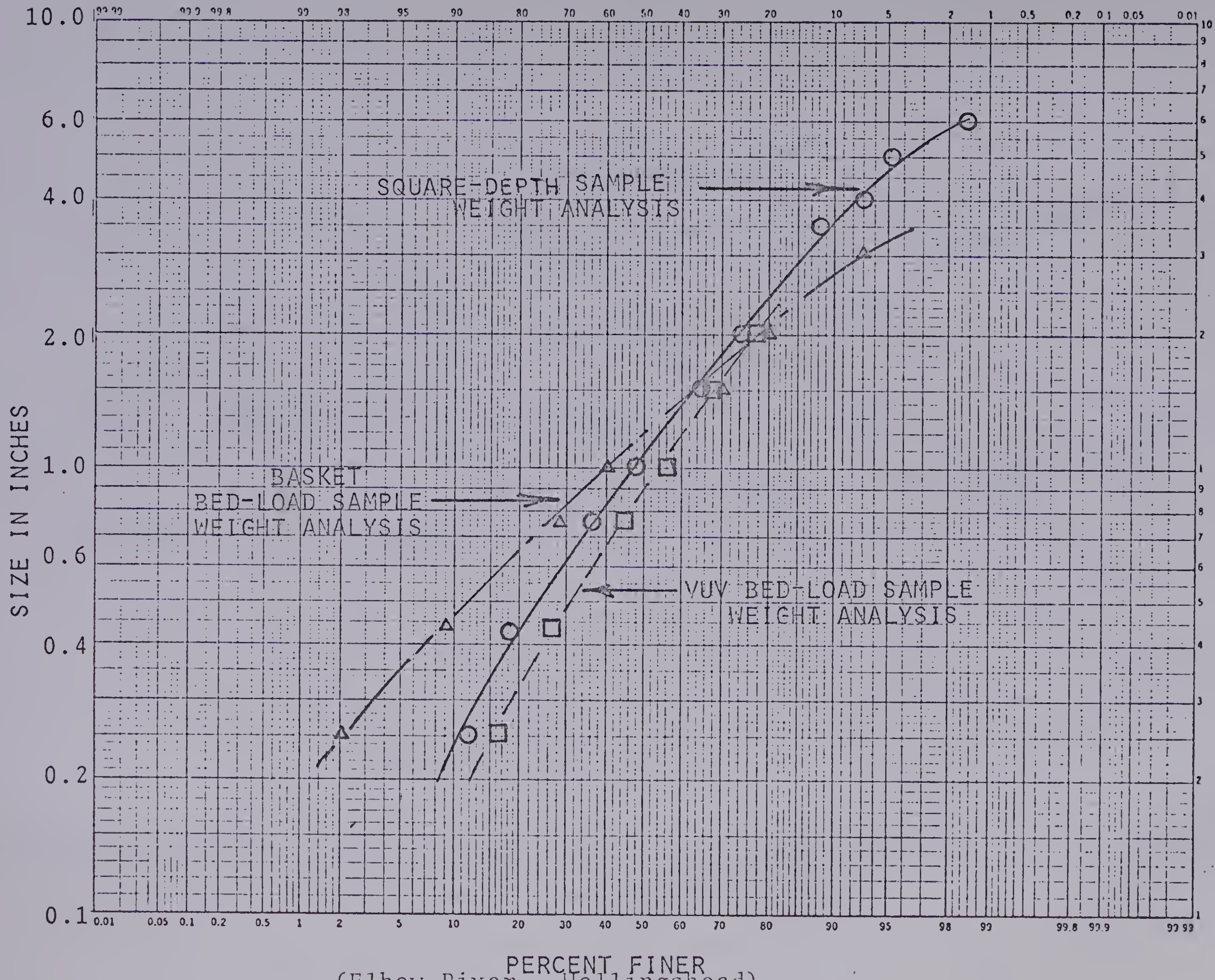
COMPARISON OF GRID SAMPLE-NUMBER ANALYSIS WITH SQUARE-SURFACE SAMPLE-VOLUME ANALYSIS

FIGURE 8



HYPOTHETICAL BED-MATERIAL DISTRIBUTION CURVES

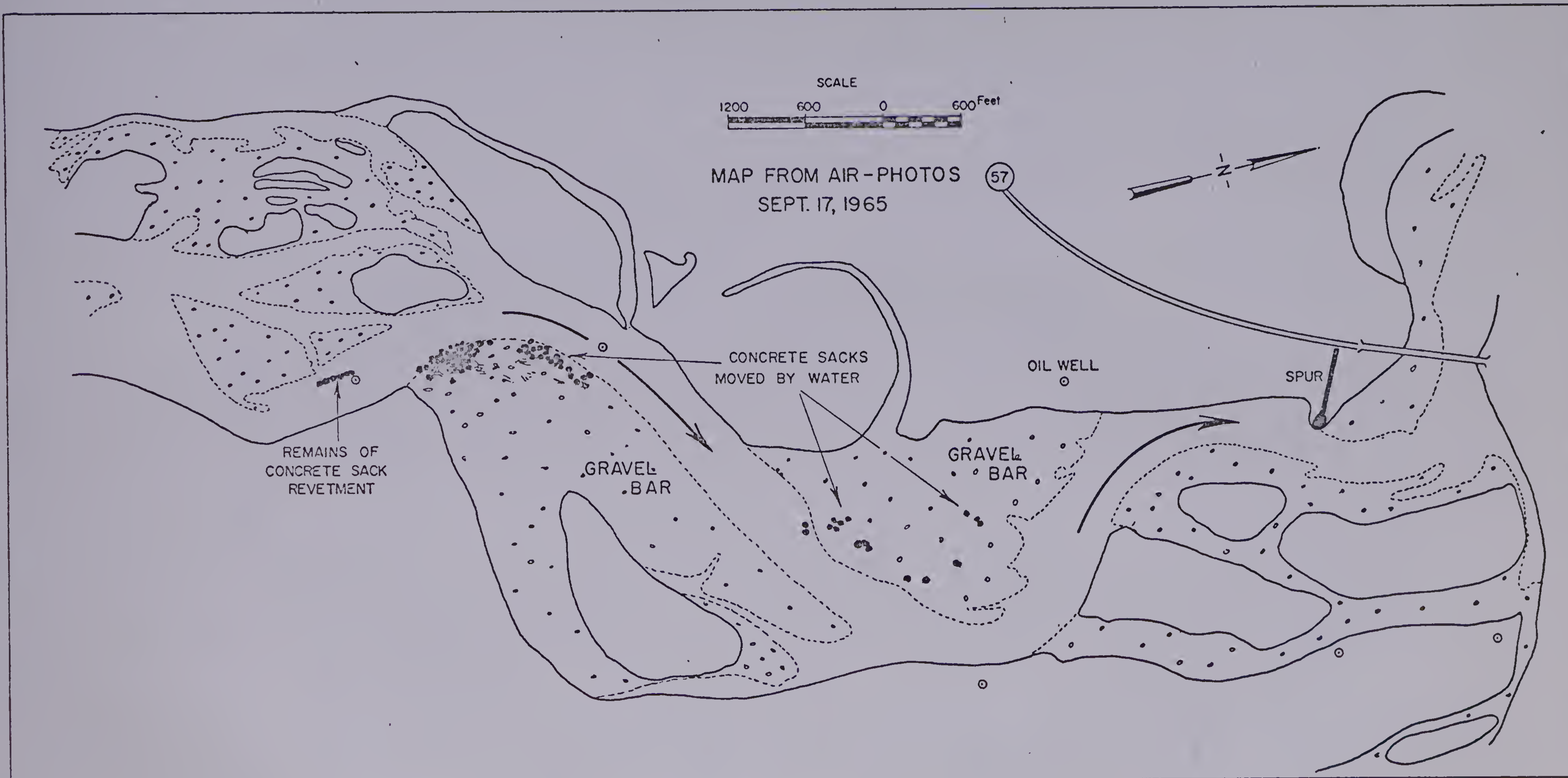
FIGURE 9



(Elbow River - Hollingshead)

COMPARISON OF DISTRIBUTION CURVES FROM BED-LOAD SAMPLES AND SQUARE--DEPTH SAMPLES

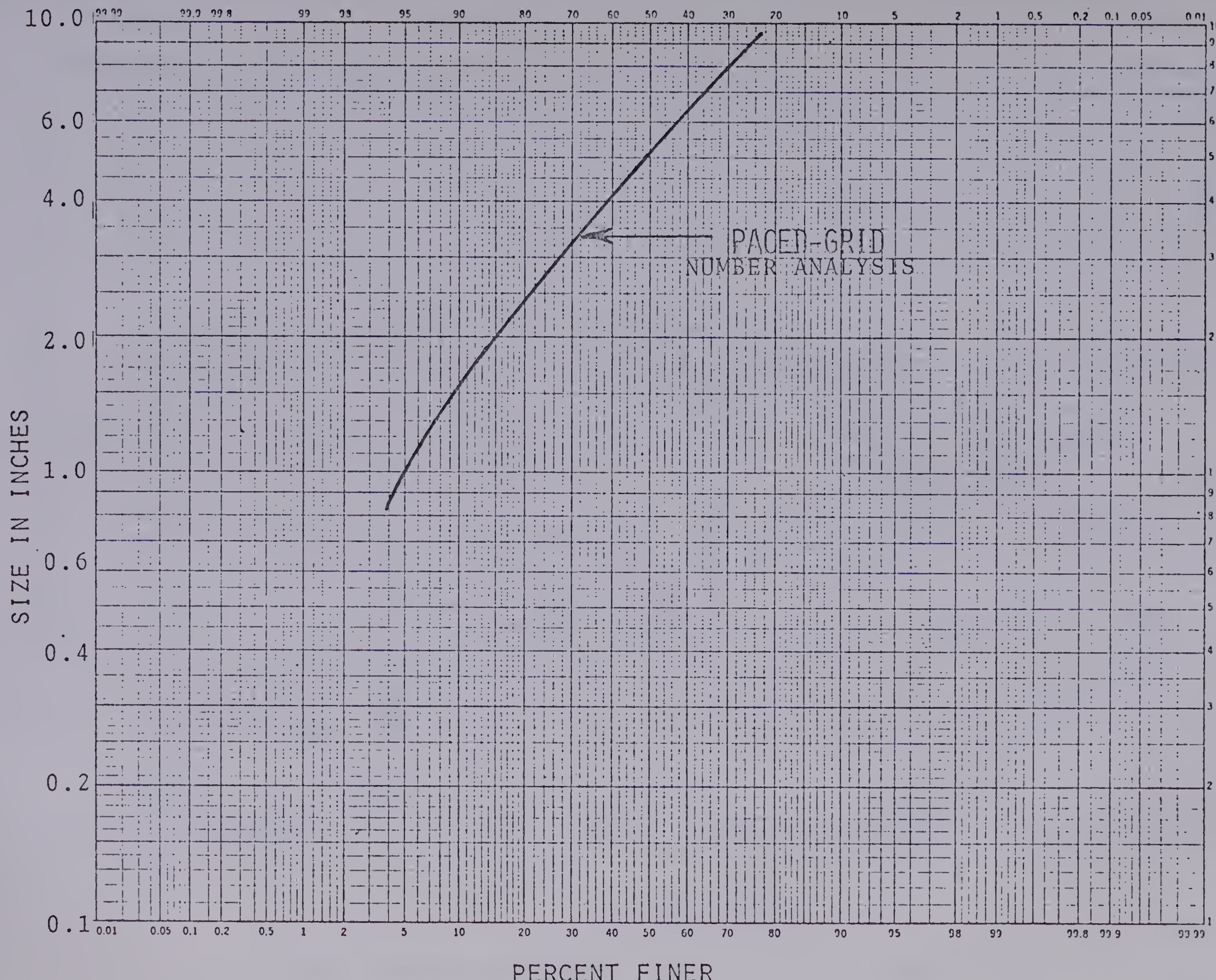
FIGURE 10



(North Sask. River- Drayton Valley)

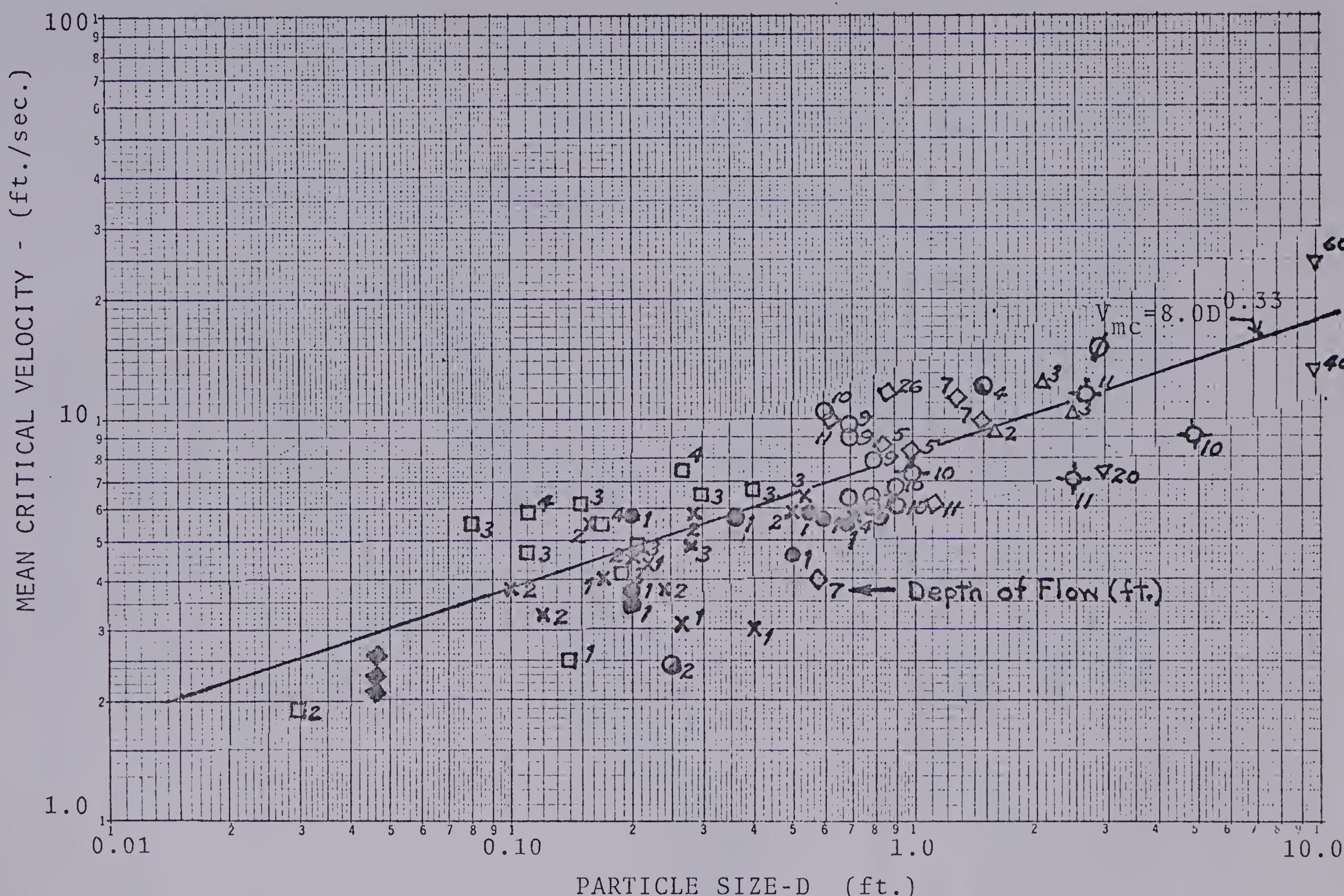
LOCATIONS OF CONCRETE-SACKS AFTER 1965 PEAK FLOW

FIGURE 11



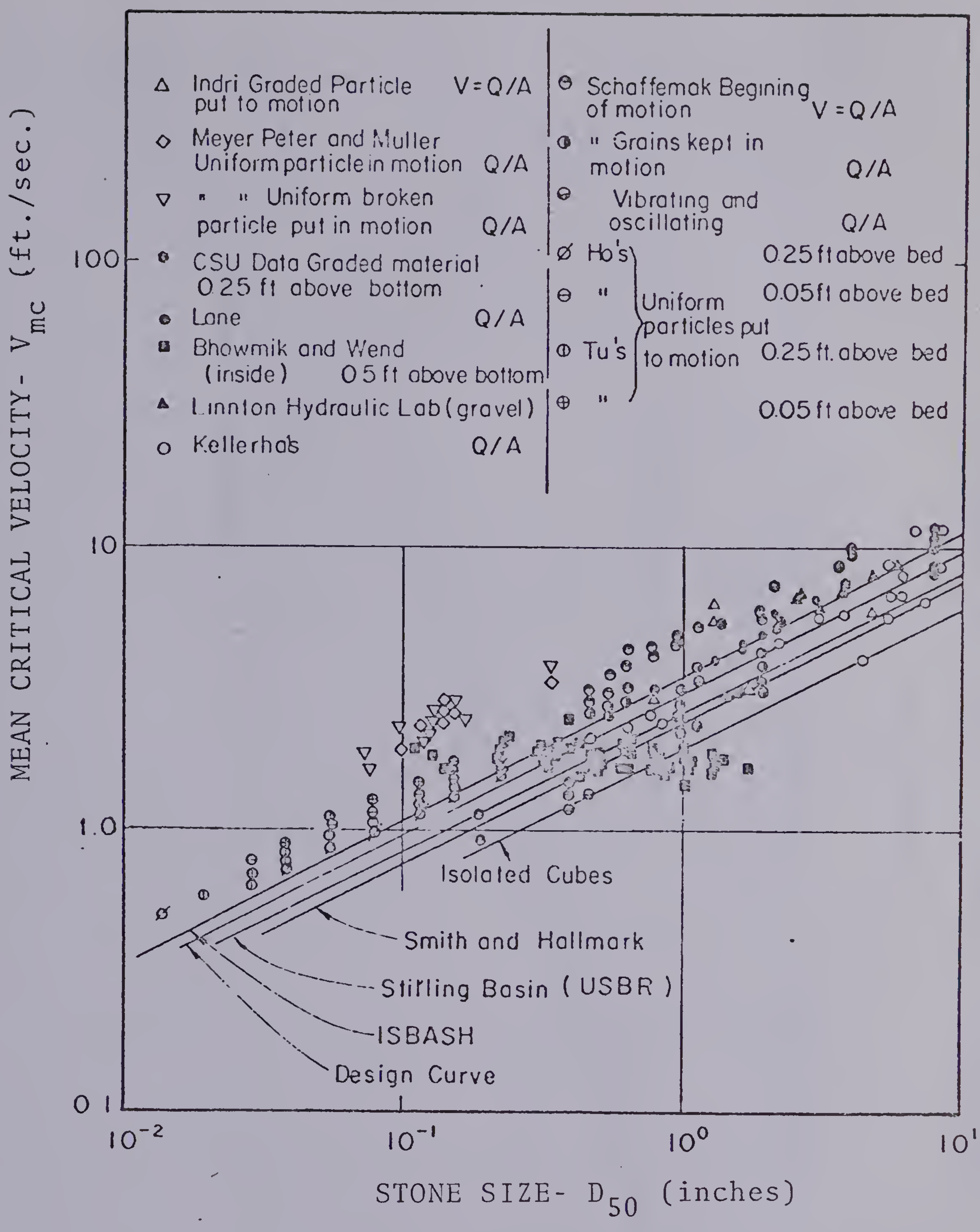
BED-MATERIAL- WILSON CREEK

FIGURE 12



MEAN CRITICAL VELOCITY VERSUS PARTICLE SIZE

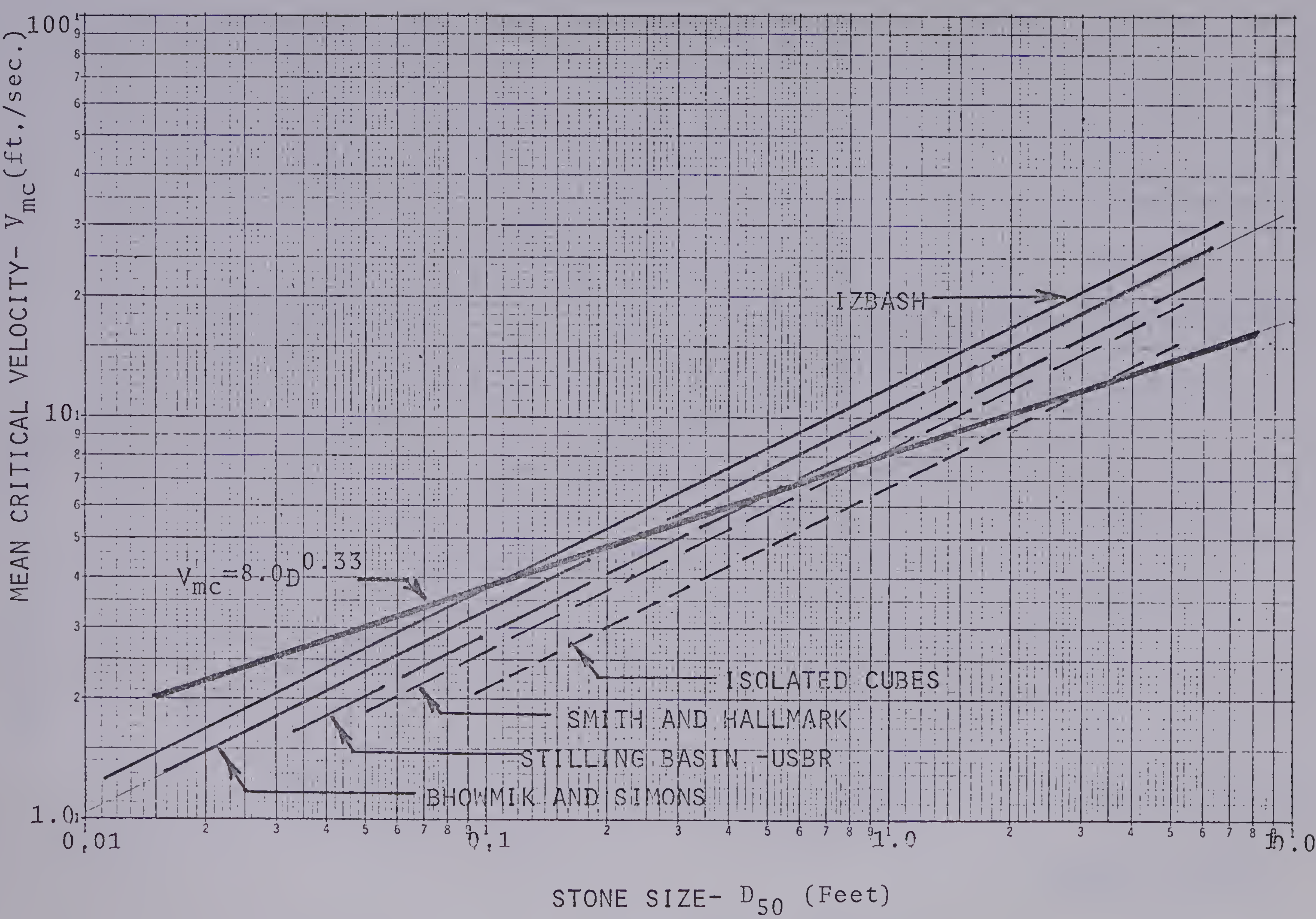
FIGURE 13



(After Bhowmik and Simons, 1970)

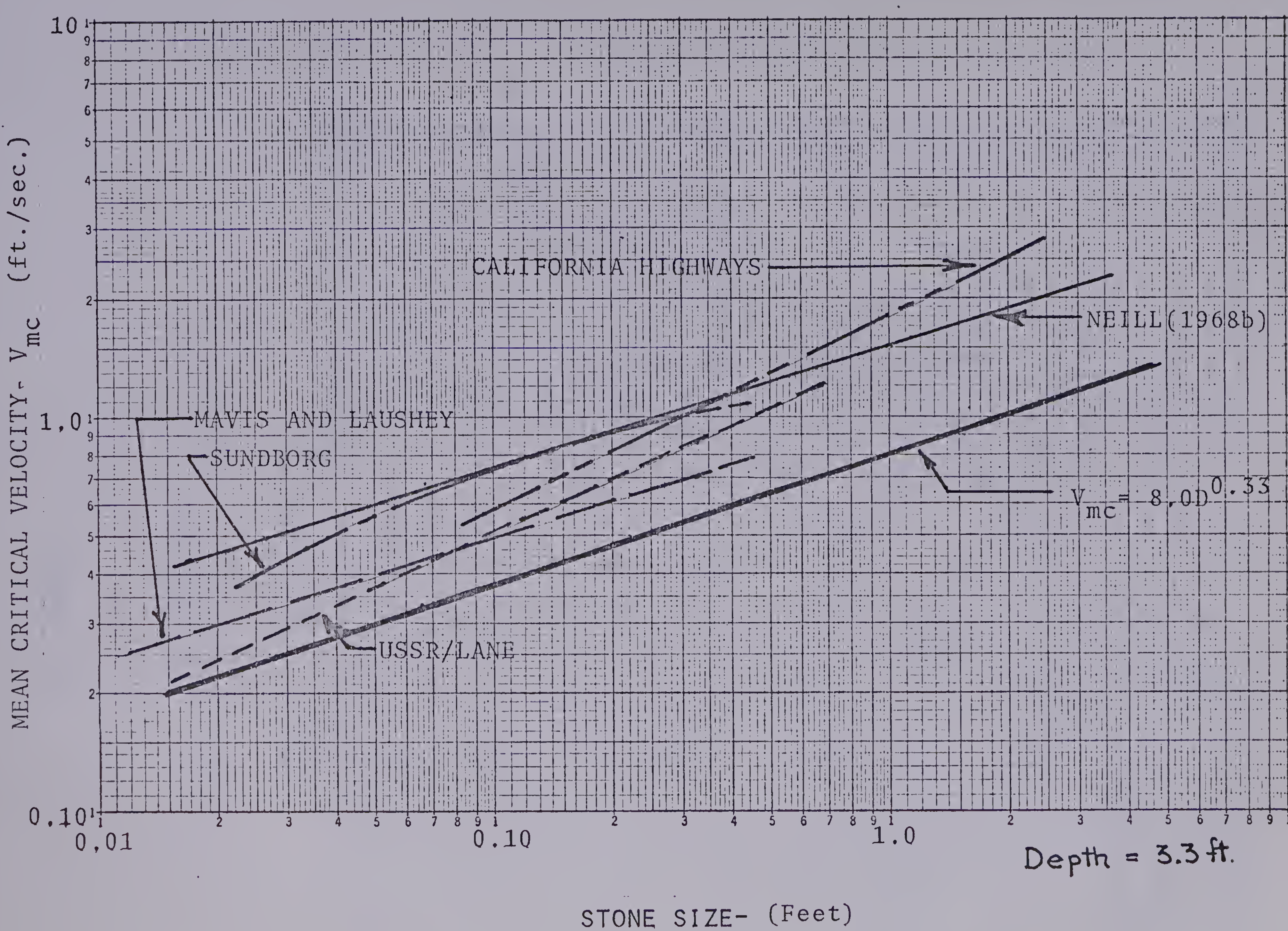
COMPARISON OF VELOCITY-STONE SIZE EQUATIONS

FIGURE 14



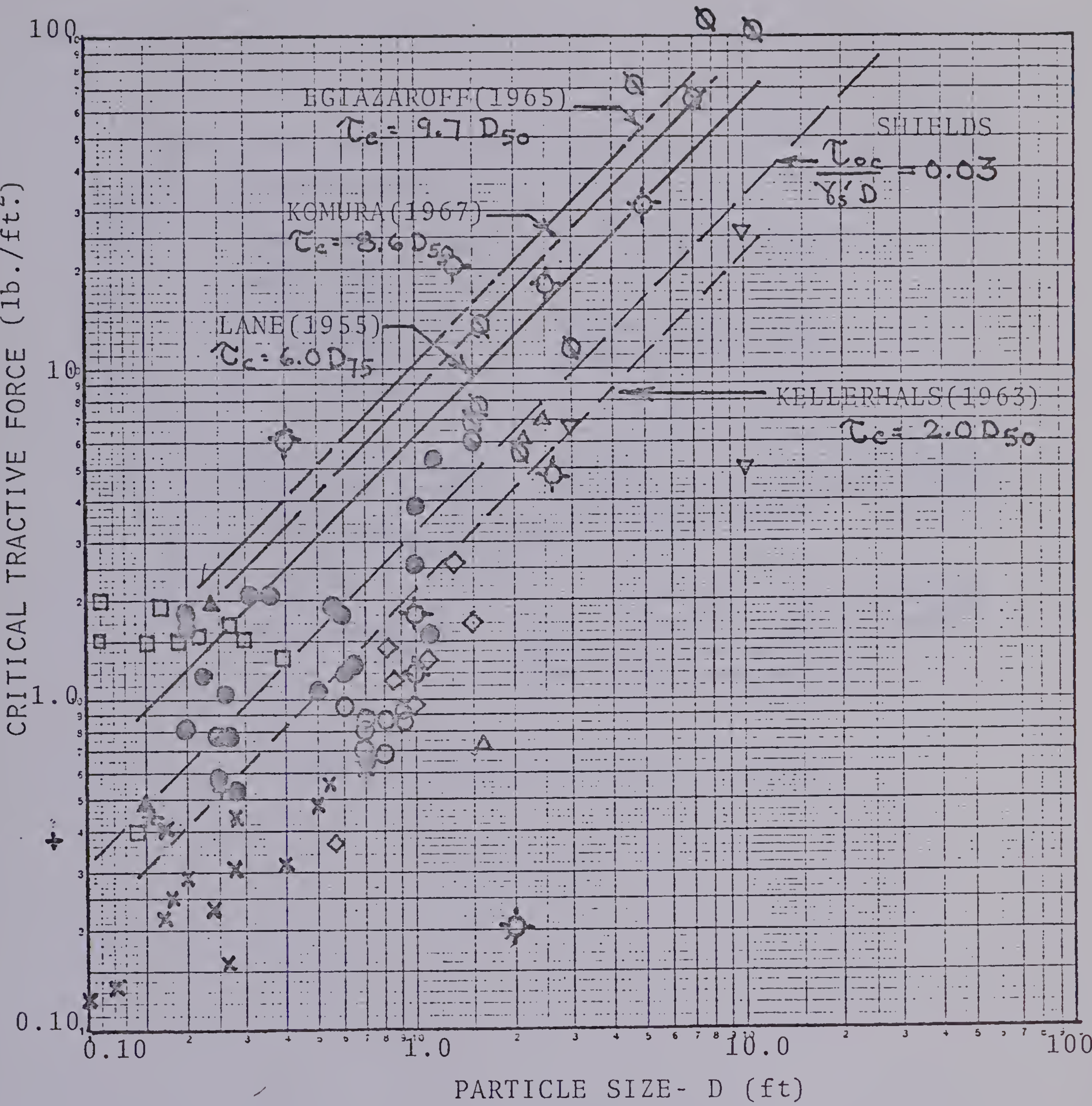
COMPARISON OF VELOCITY- STONE SIZE EQUATIONS

FIGURE 15



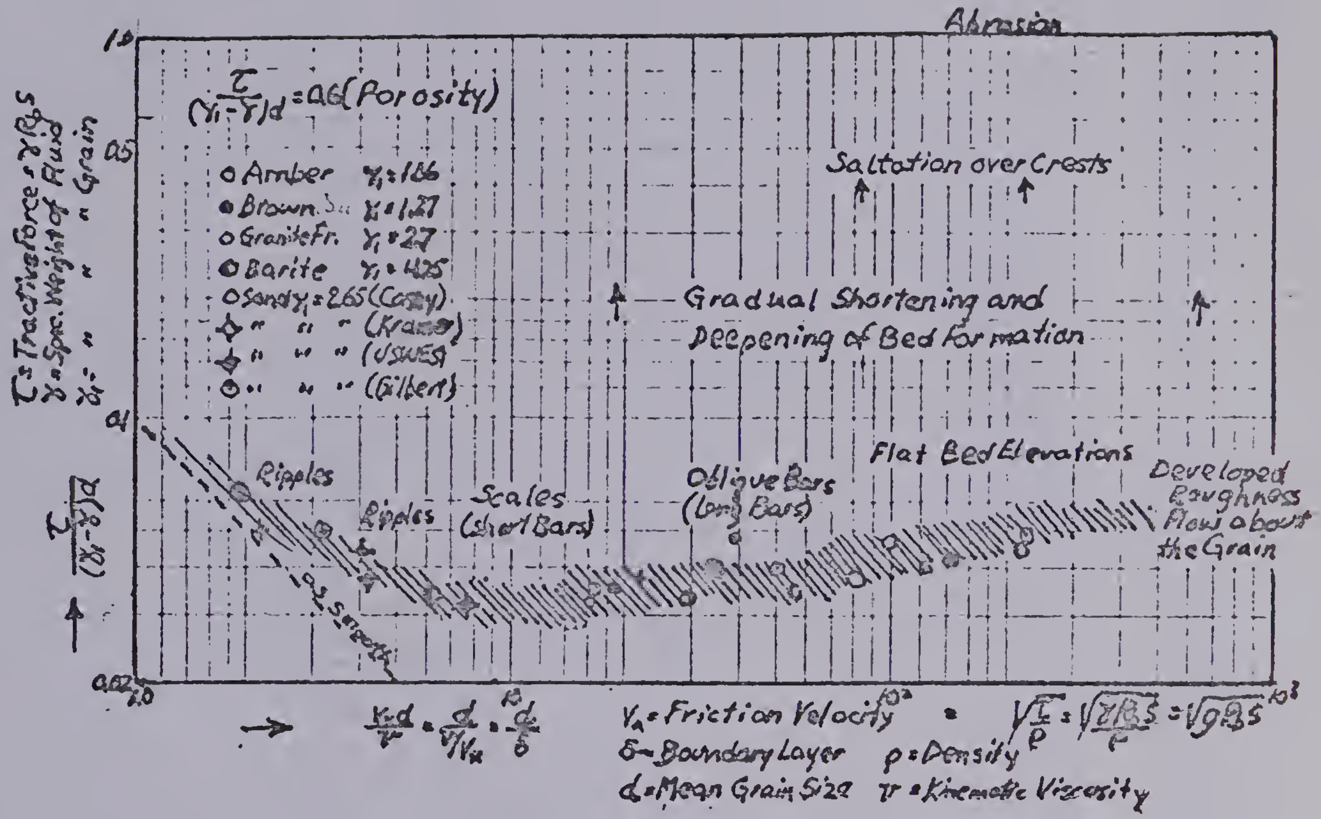
COMPARISON OF VELOCITY- STONE SIZE EQUATIONS

FIGURE 16



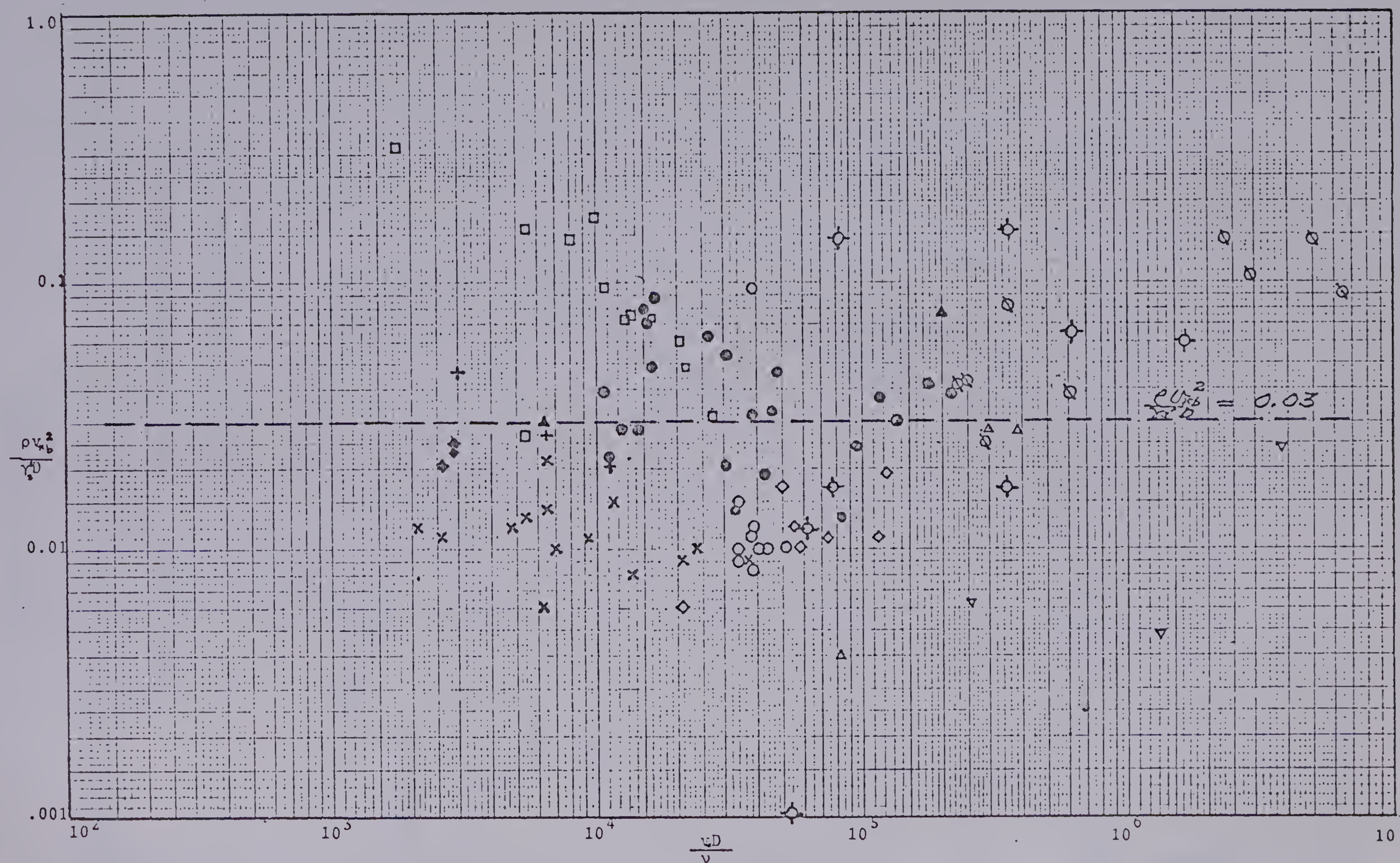
CRITICAL TRACTIVE FORCE VERSUS
PARTICLE SIZE

FIGURE 17



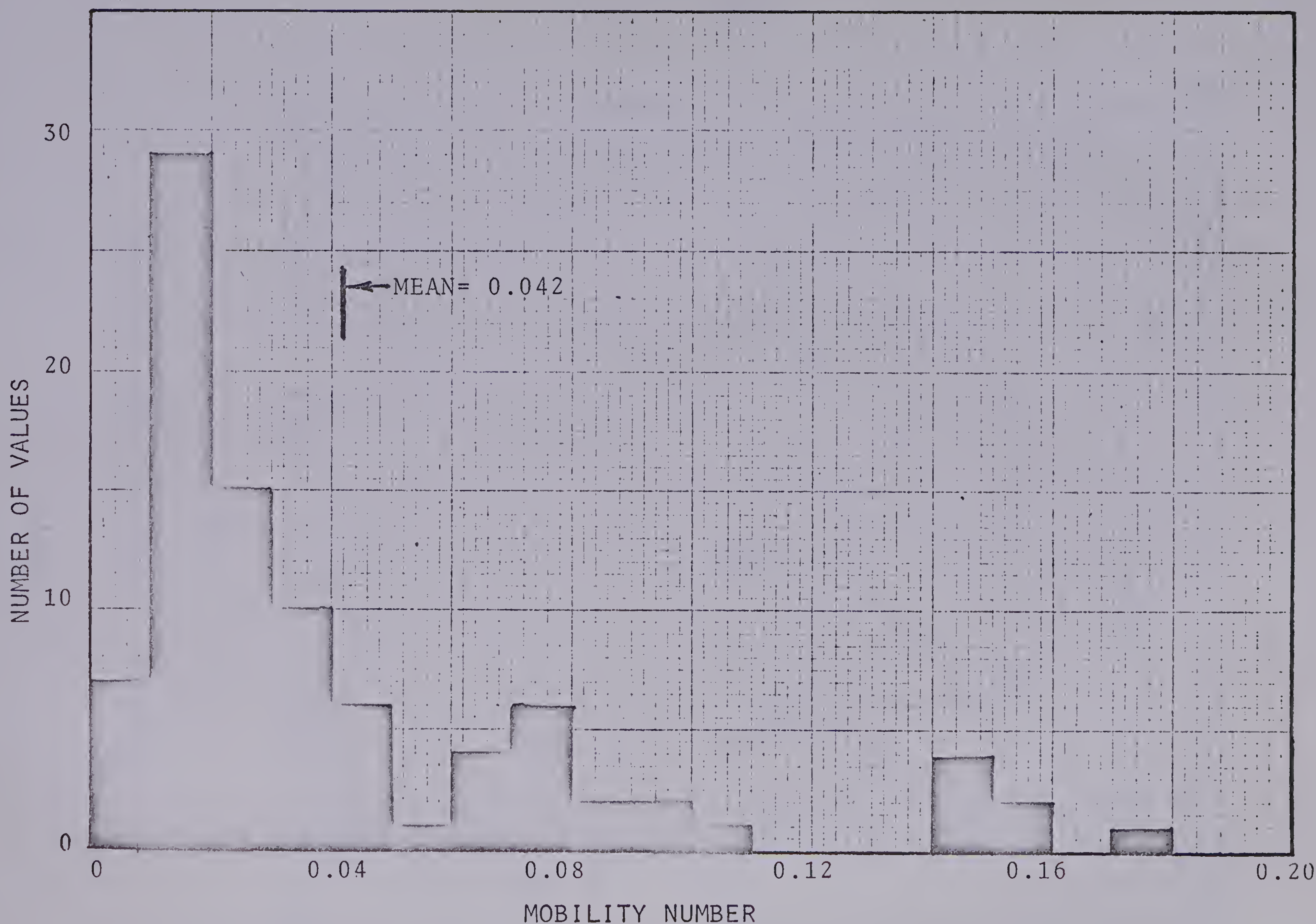
COPY OF SHIELDS' ORIGINAL DIAGRAM

FIGURE 18



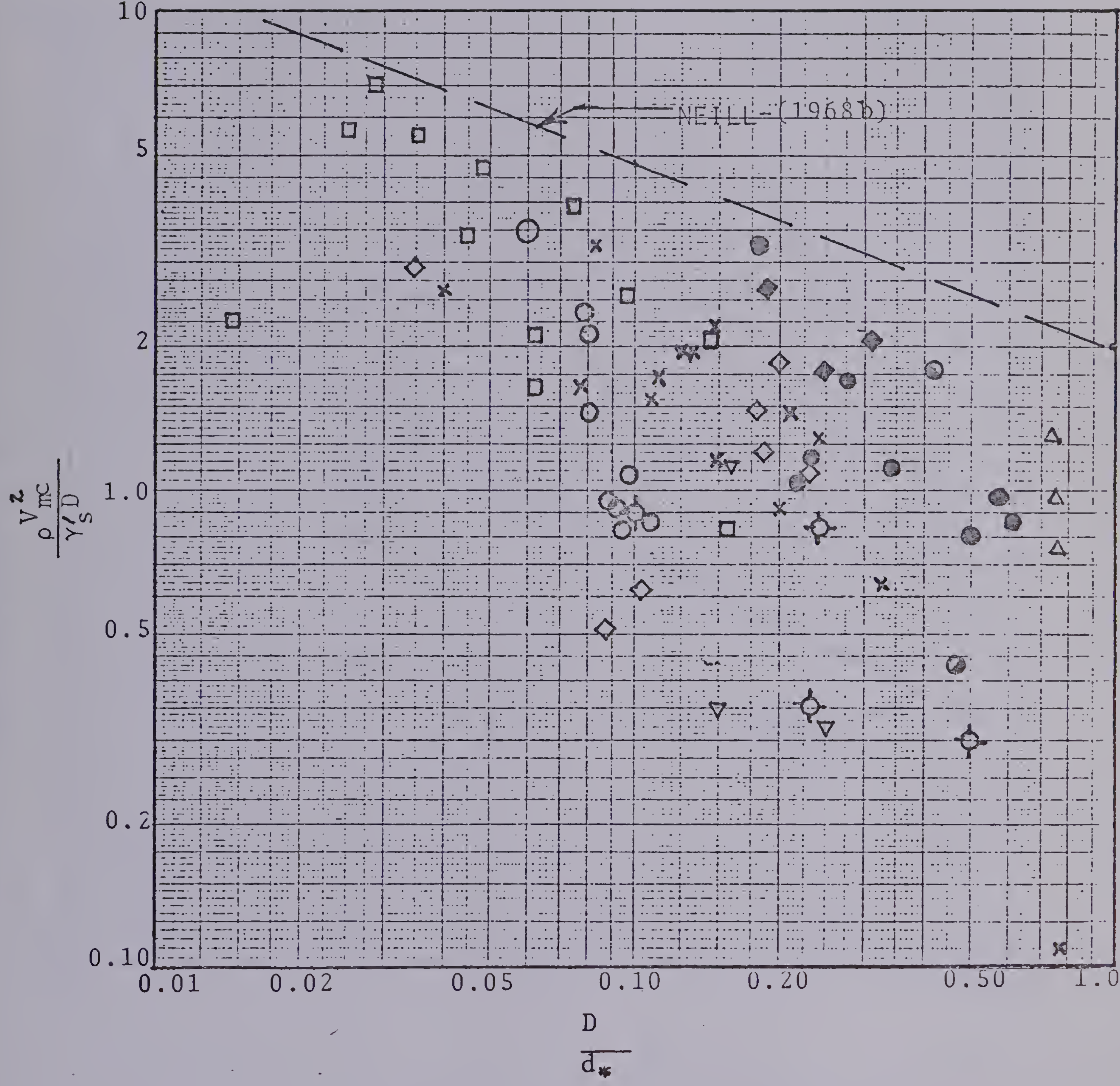
MOBILITY NUMBER VERSUS PARTICLE REYNOLDS NUMBER

FIGURE 19



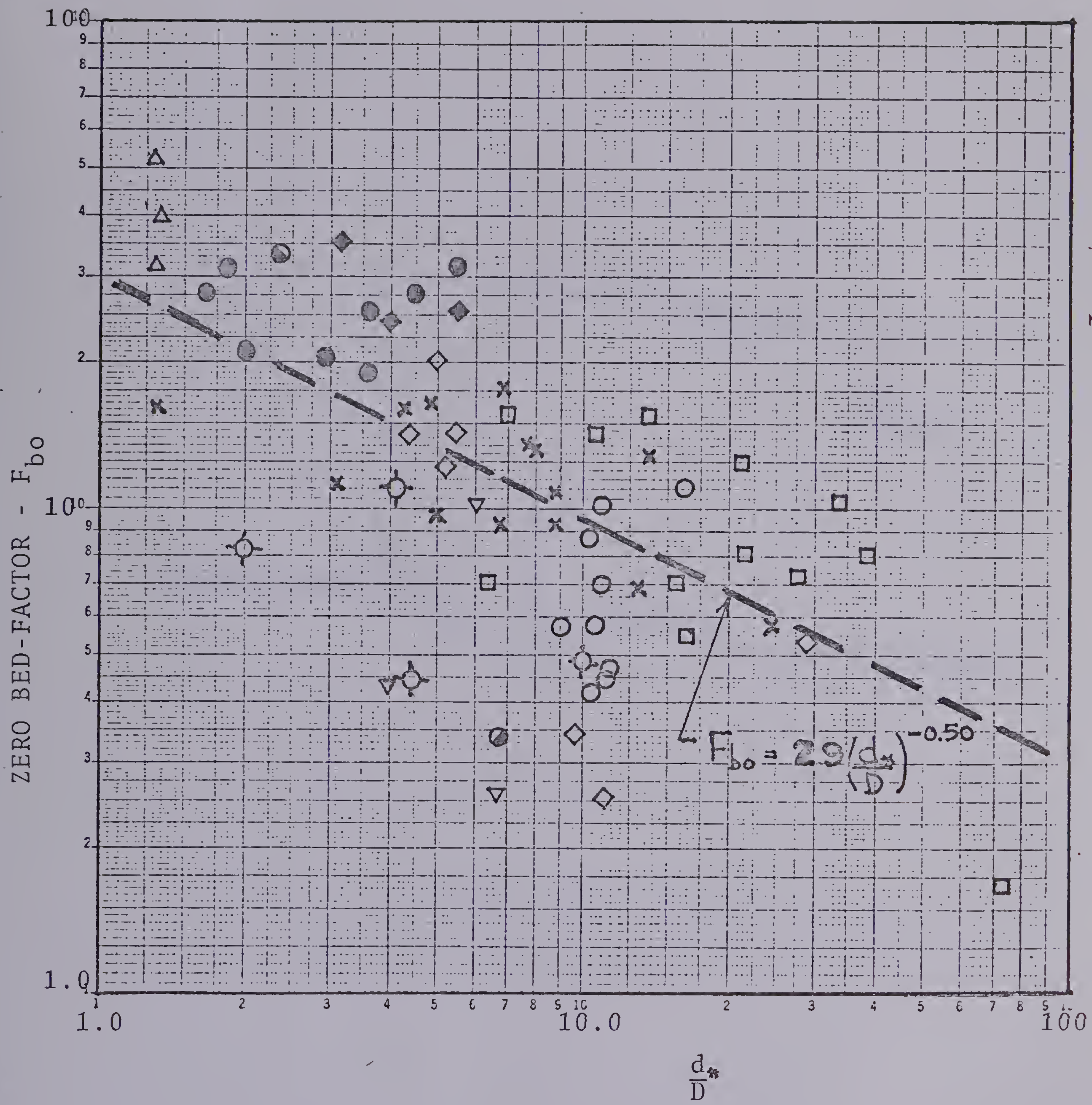
HISTOGRAM OF FIELD MOBILITY NUMBERS

FIGURE 20



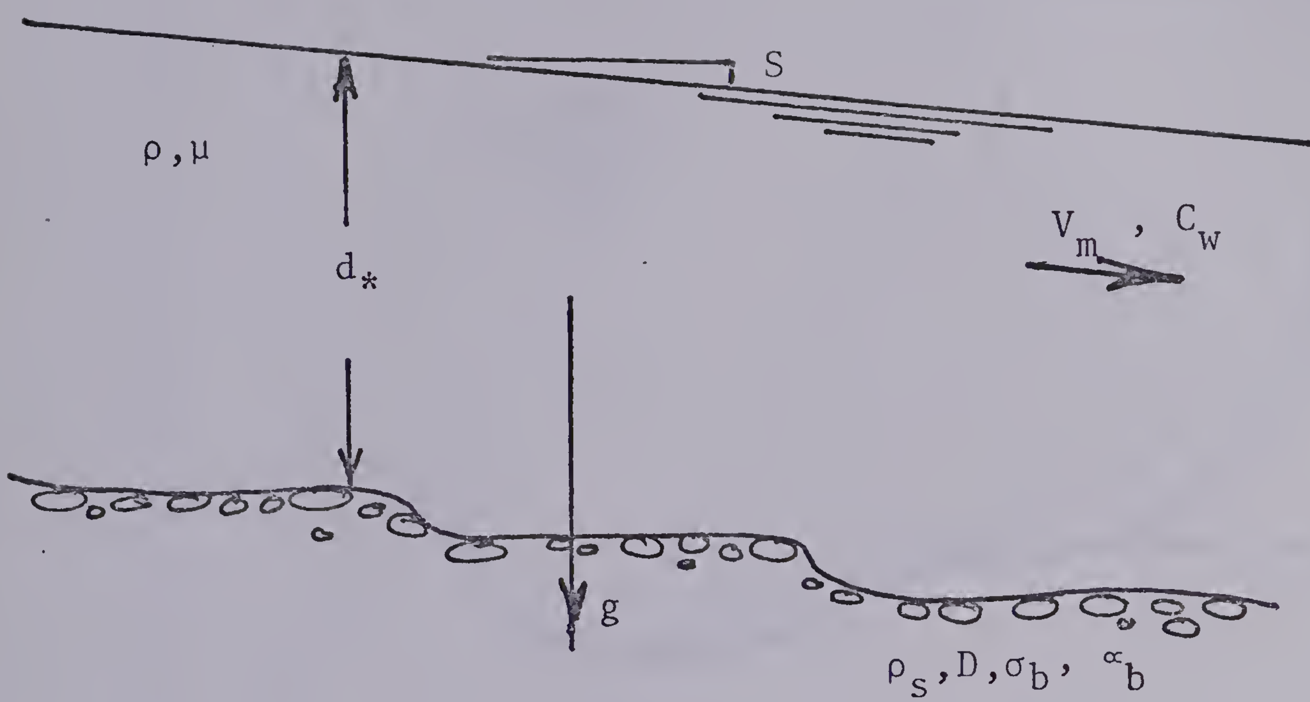
MODIFIED MOBILITY NUMBER VERSUS $\frac{D}{d_*}$

FIGURE 21



F_{bo} VERSUS $\frac{d_*}{D}$ (AFTER BLENCH 1969)

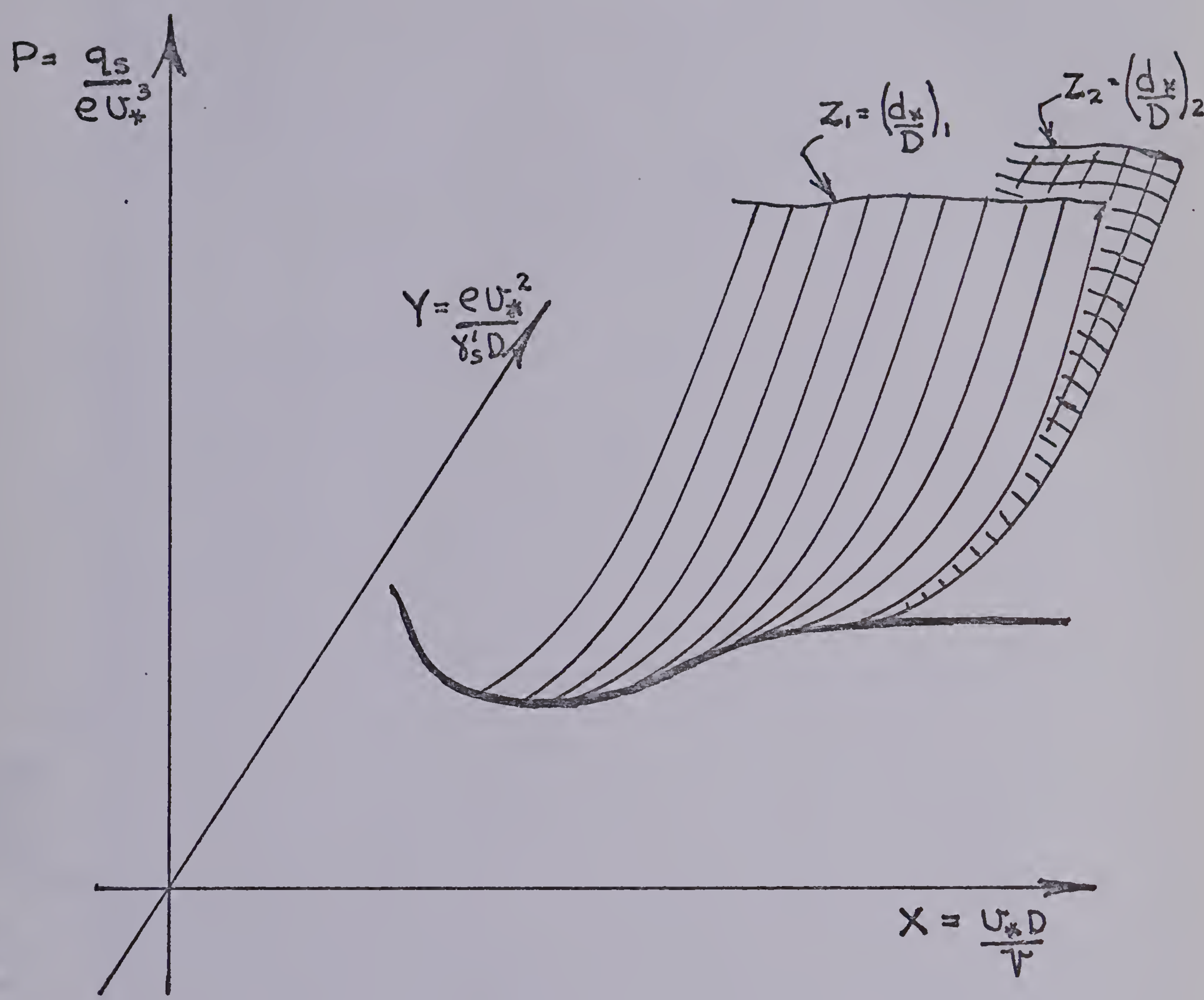
FIGURE 22



Note- width is more than 5 times mean depth

INDEPENDENT VARIABLES
FOR THE TRANSPORT OF SEDIMENT IN OPEN
CHANNELS

FIGURE 23



THREE DIMENSIONAL SEDIMENT TRANSPORT PLOT

FIGURE 24

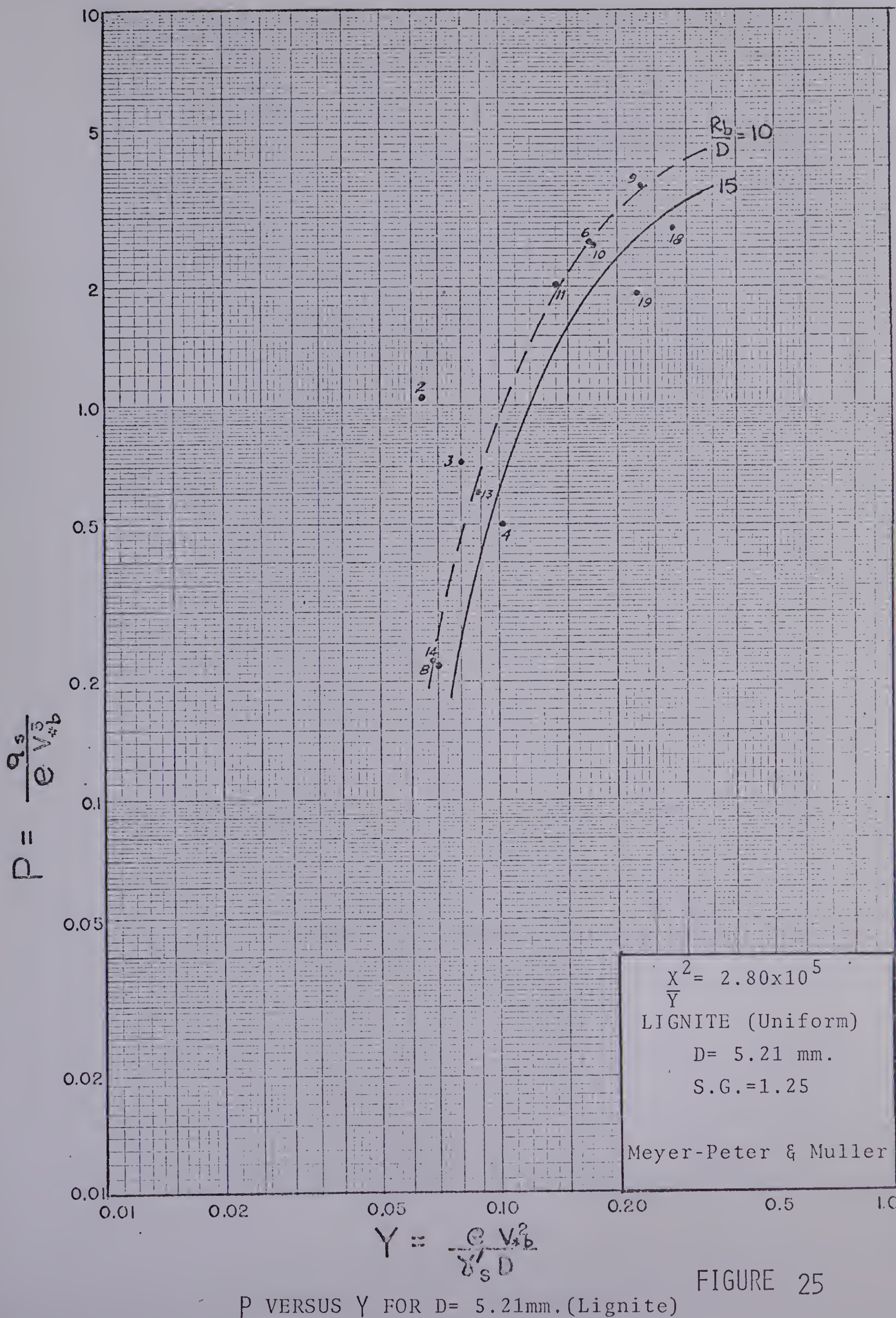


FIGURE 25

P VERSUS Y FOR D= 5.21mm.(Lignite)

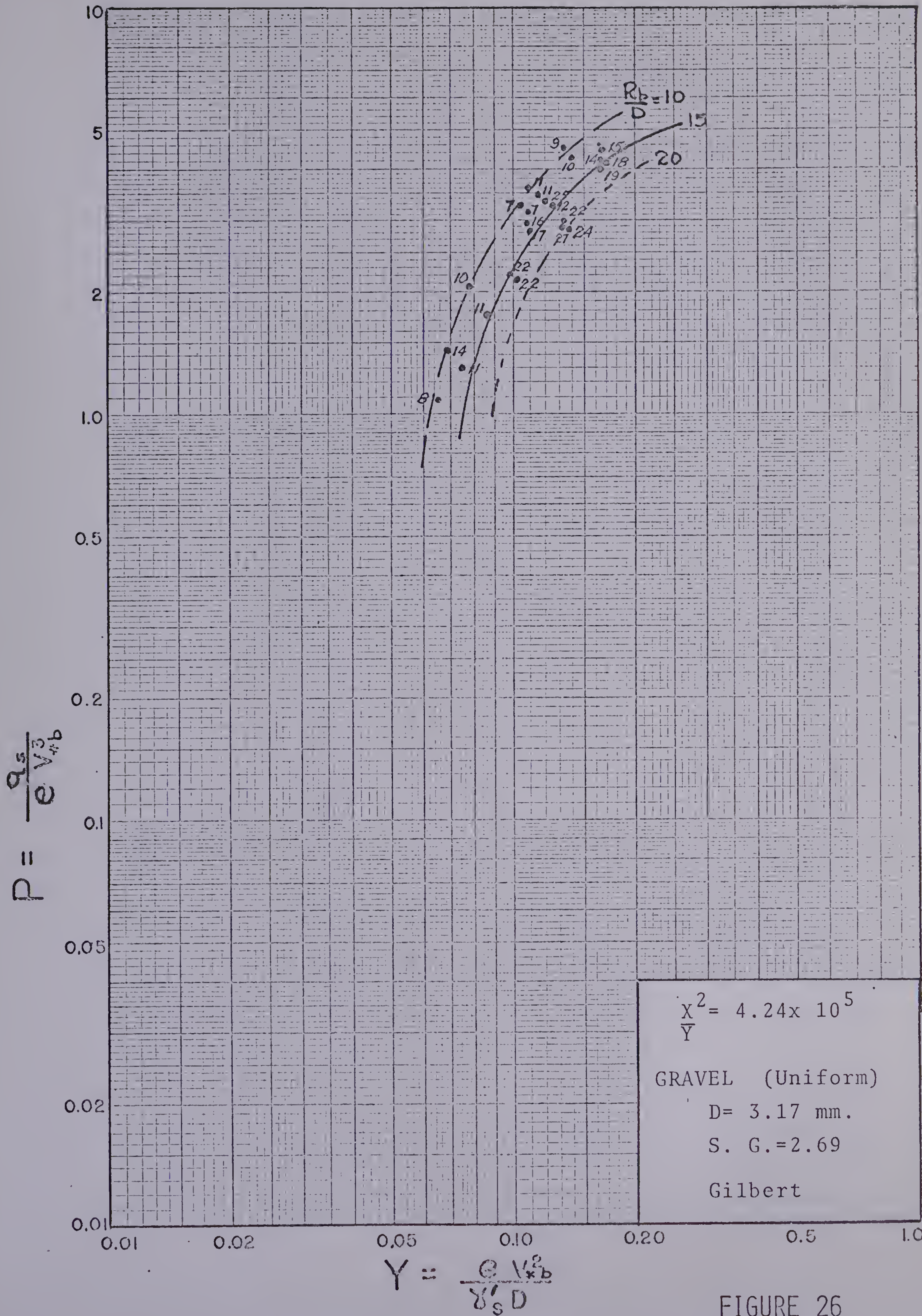


FIGURE 26

P VERSUS Y D= 3.17mm.(Gravel)

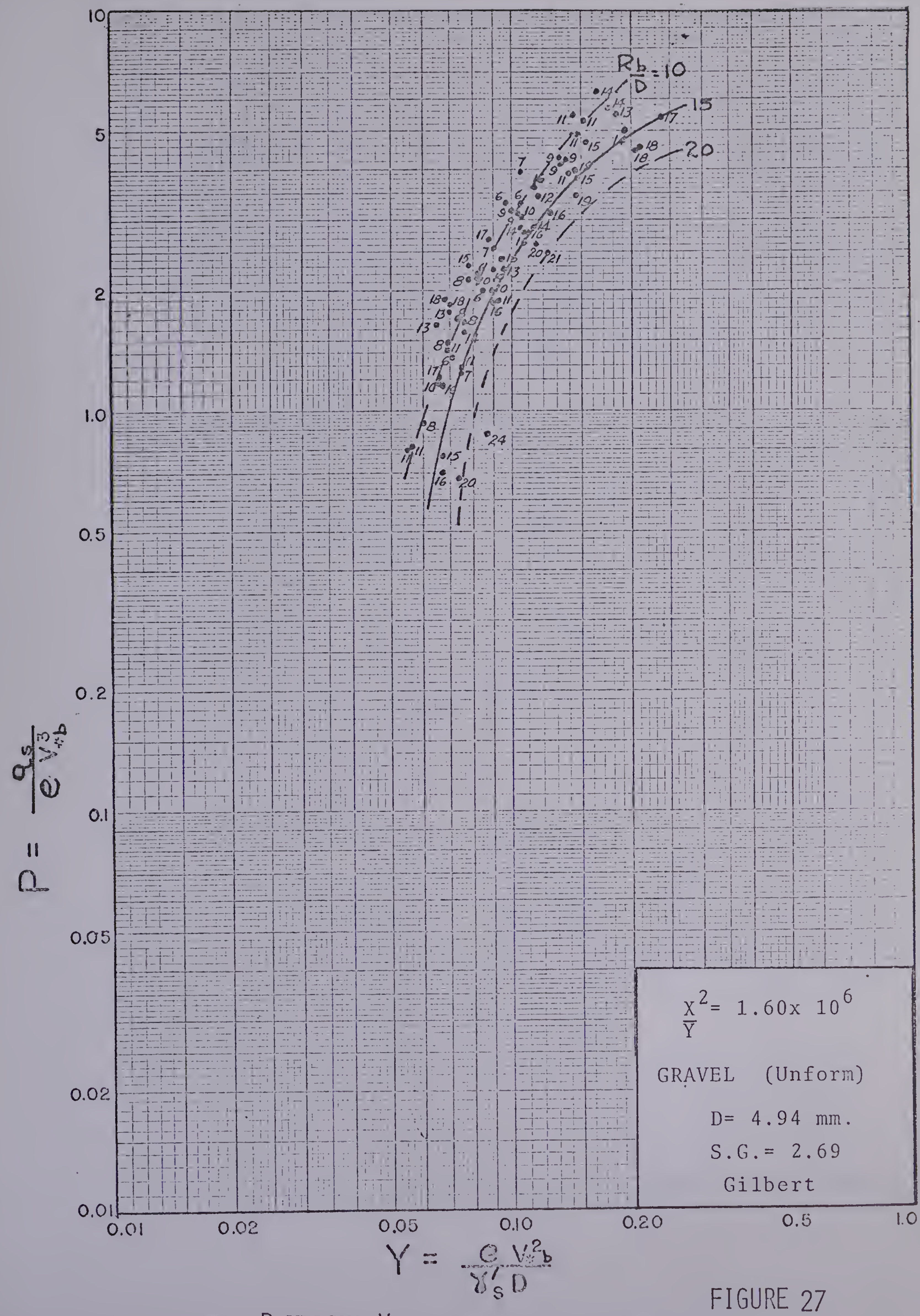


FIGURE 27

P VERSUS Y FOR D=4.94mm.(Gravel)

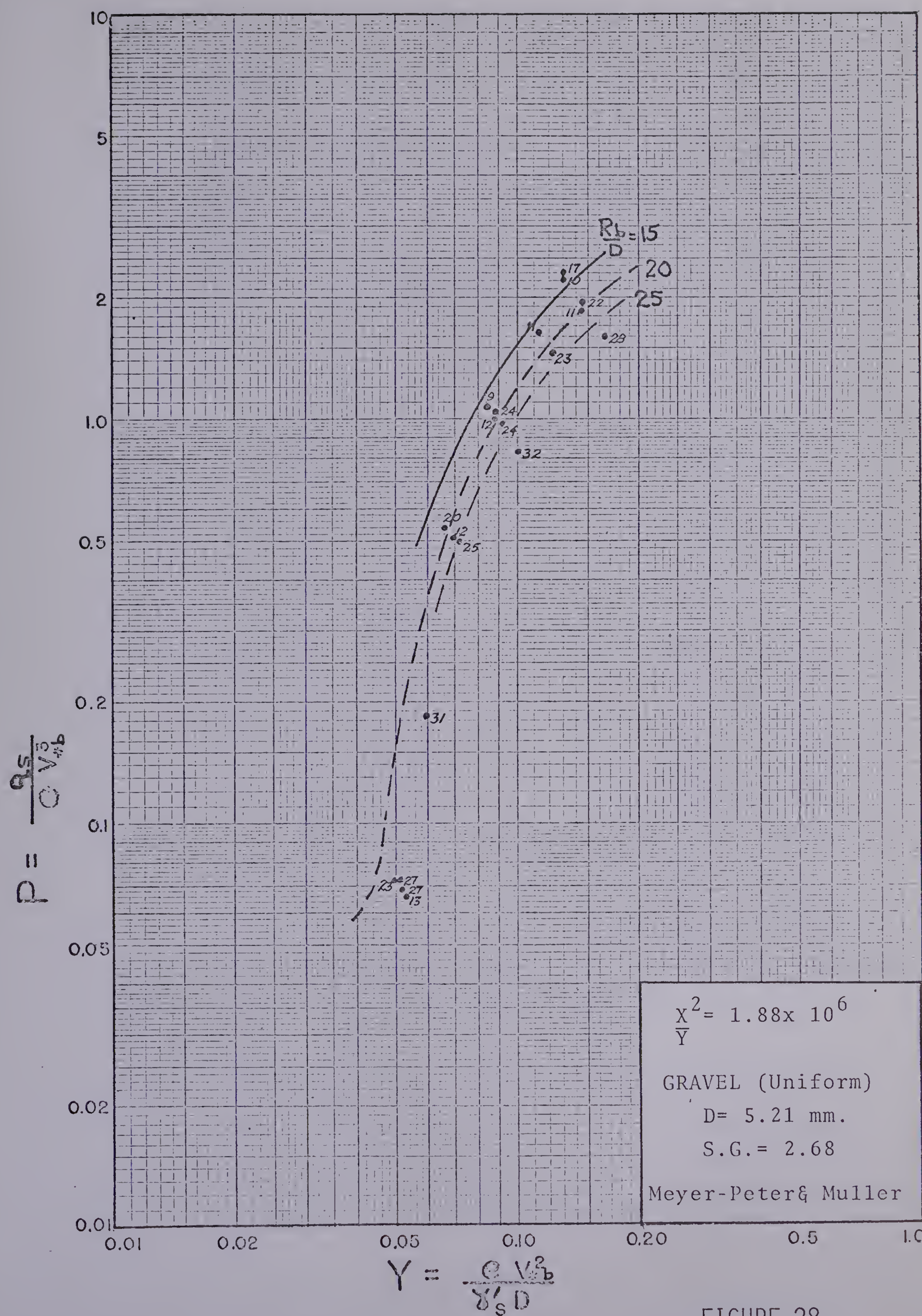


FIGURE 28

P VERSUS Y FOR D=5.21mm.(Gravel)

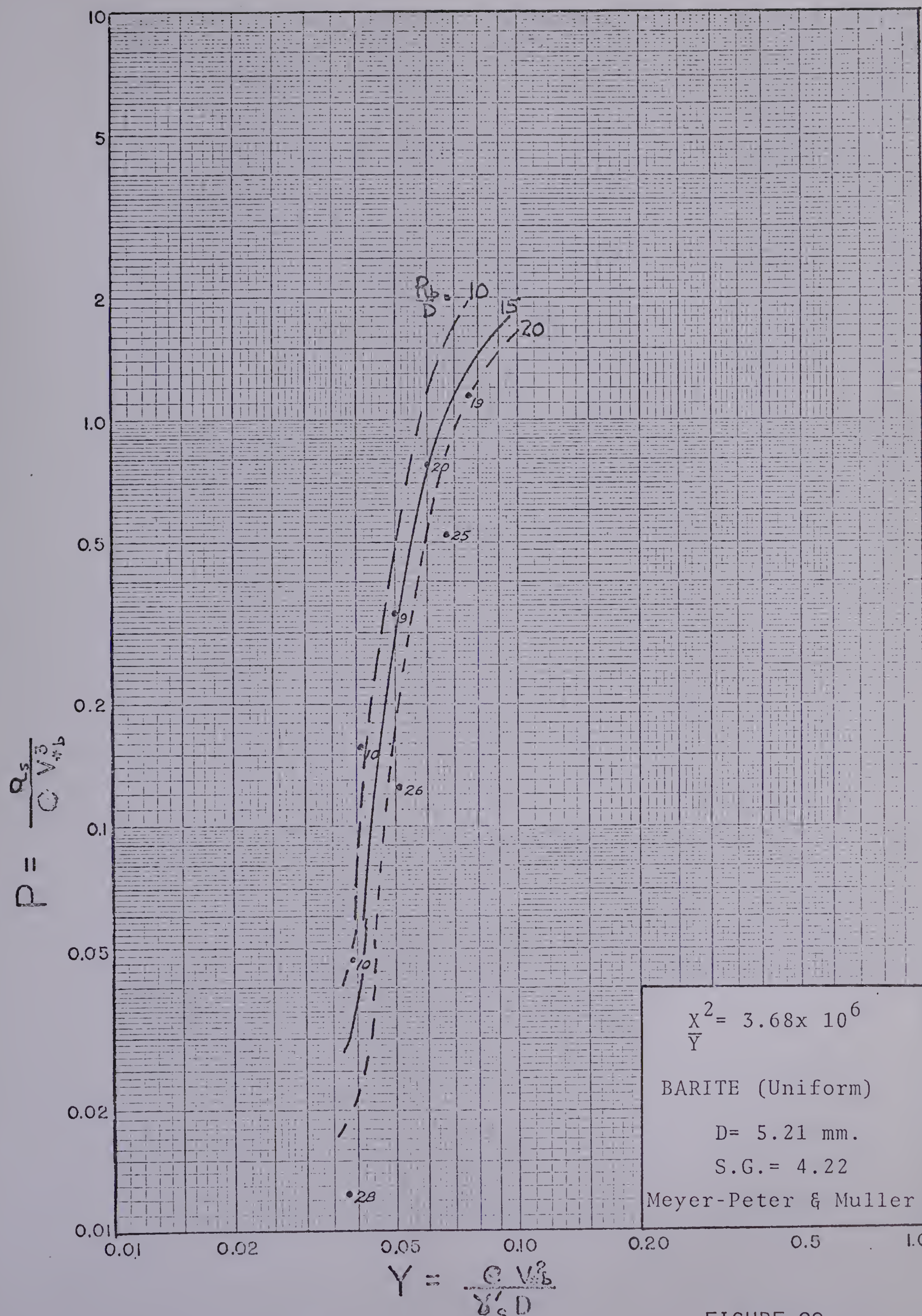


FIGURE 29

P VERSUS Y FOR D=5.21mm.(Barite)

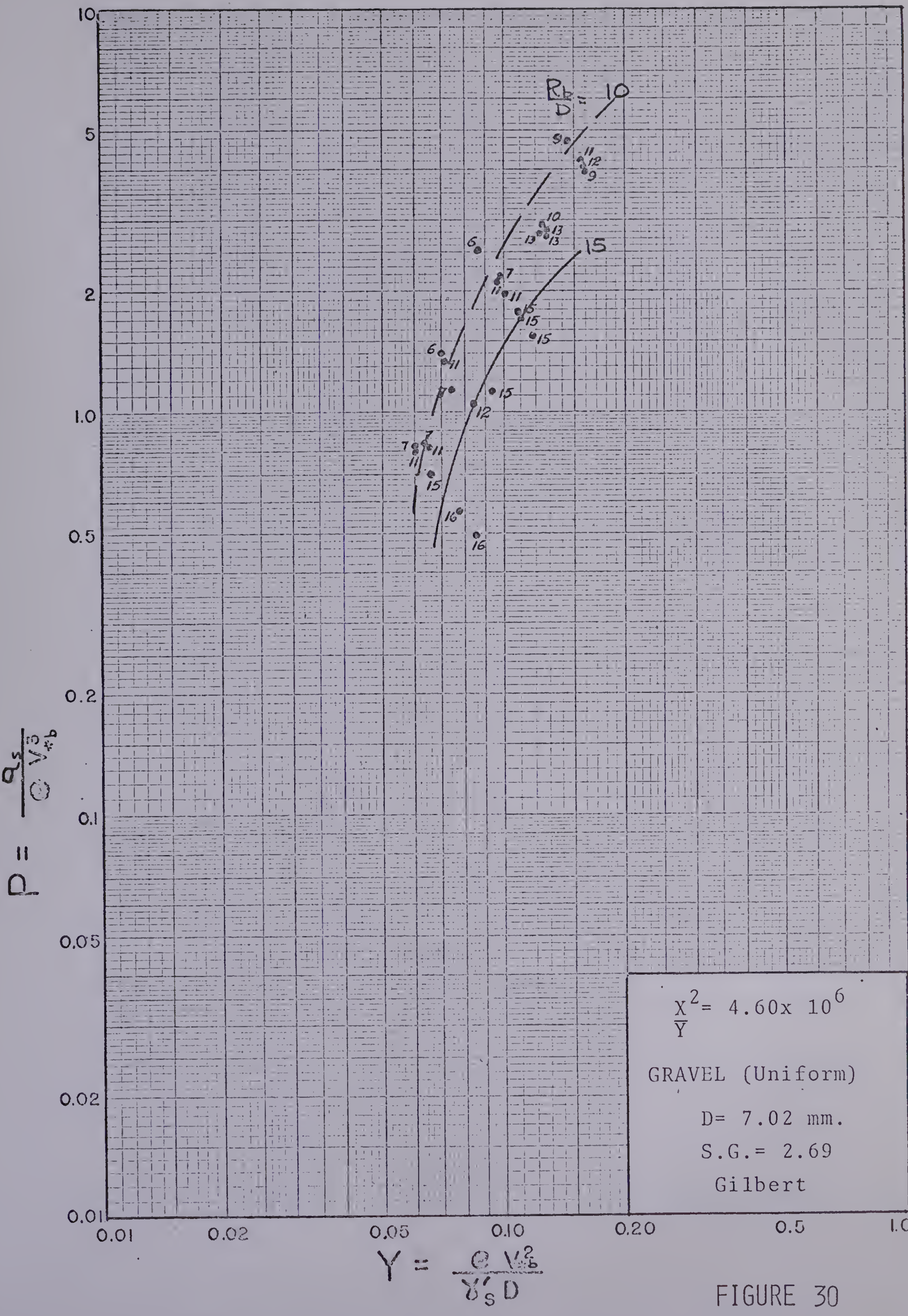
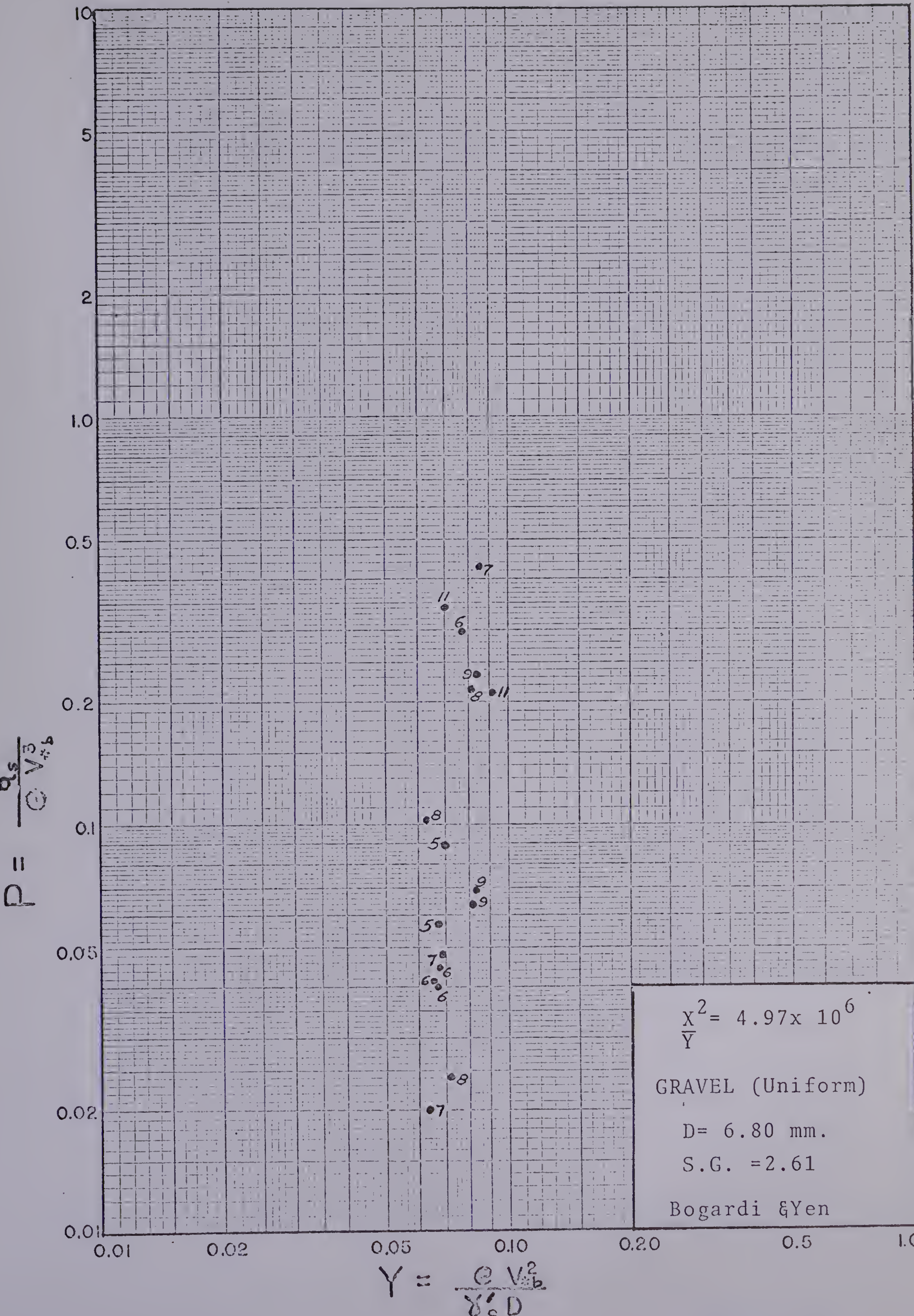


FIGURE 30

P VERSUS Y FOR D=7.02mm.(Gravel)



P VERSUS Y FOR D=6.80mm.(Gravel)

FIGURE 31

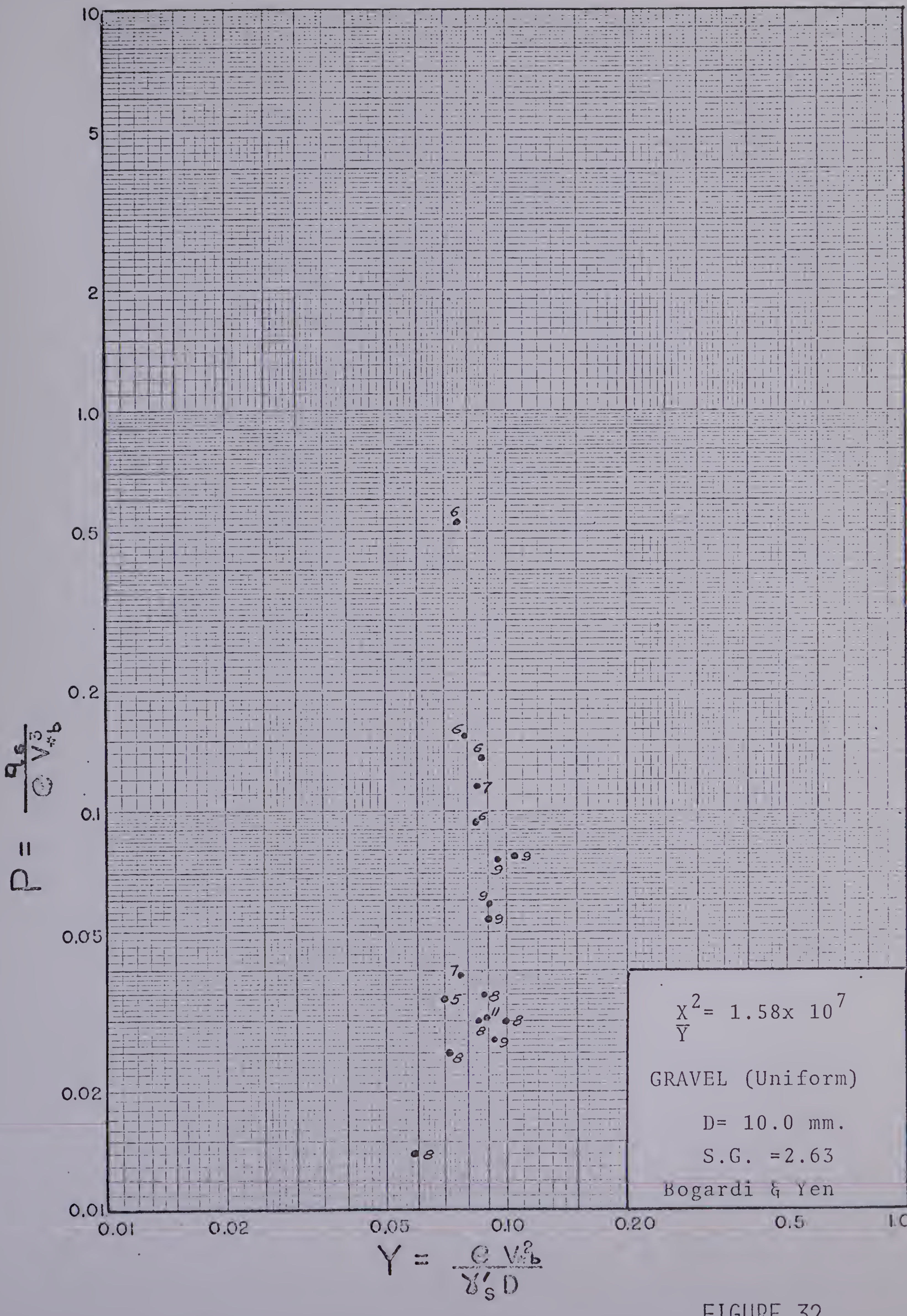


FIGURE 32

P VERSUS Y FOR D=10.0mm.(Gravel)

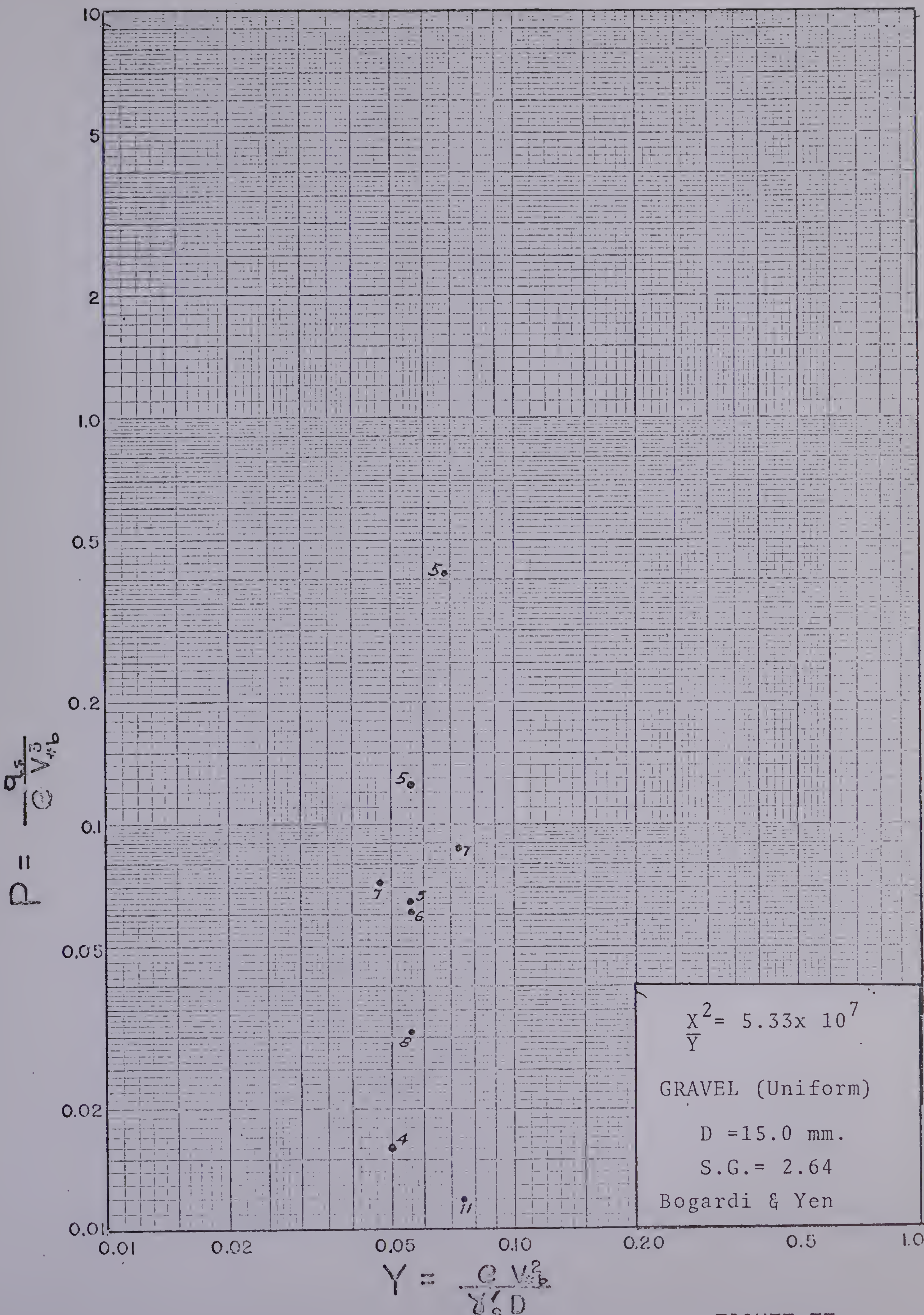


FIGURE 33

P VERSUS Y FOR D=15.0mm.(Gravel)

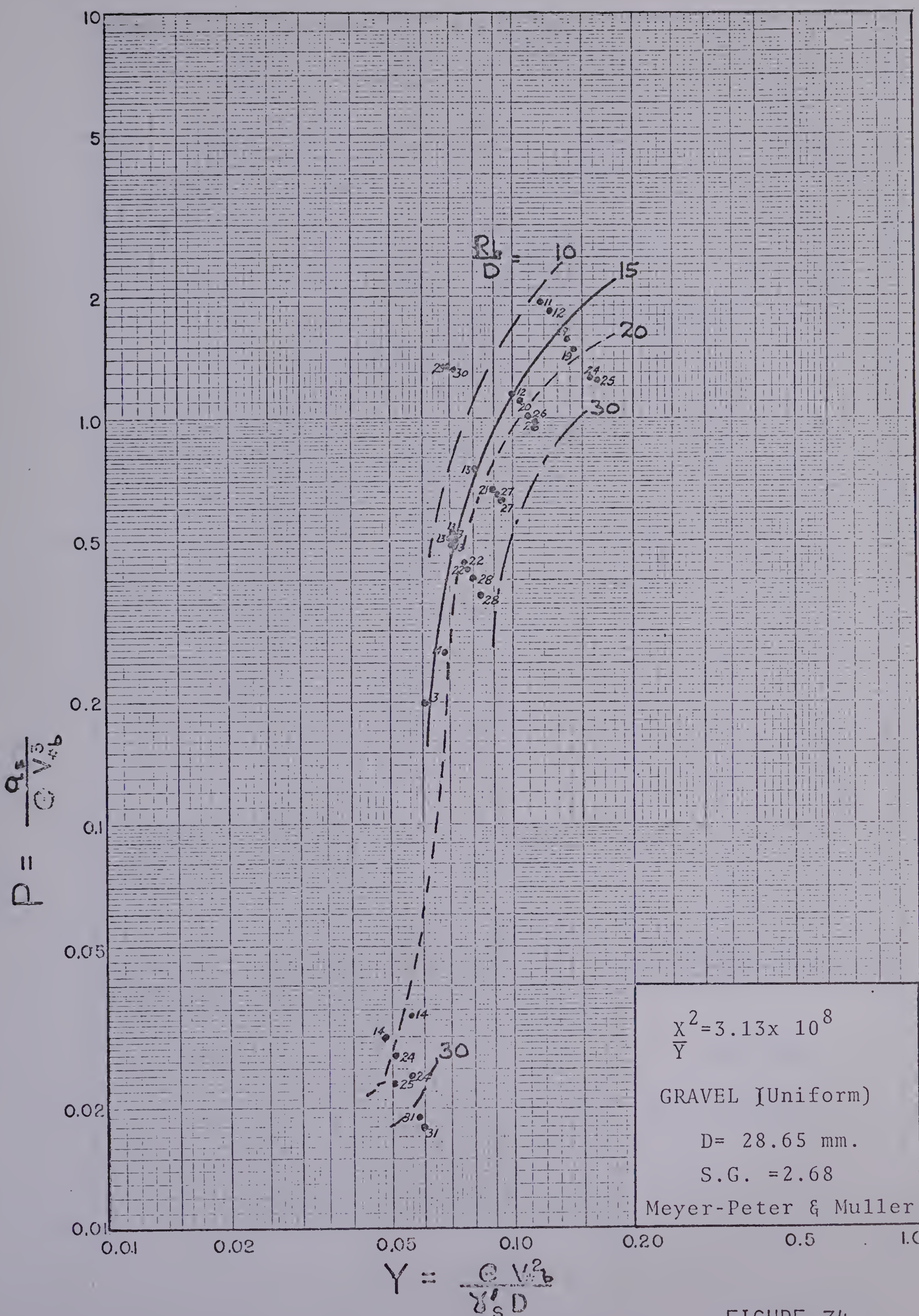


FIGURE 34

P VERSUS Y FOR D=28.65mm.(Gravel)

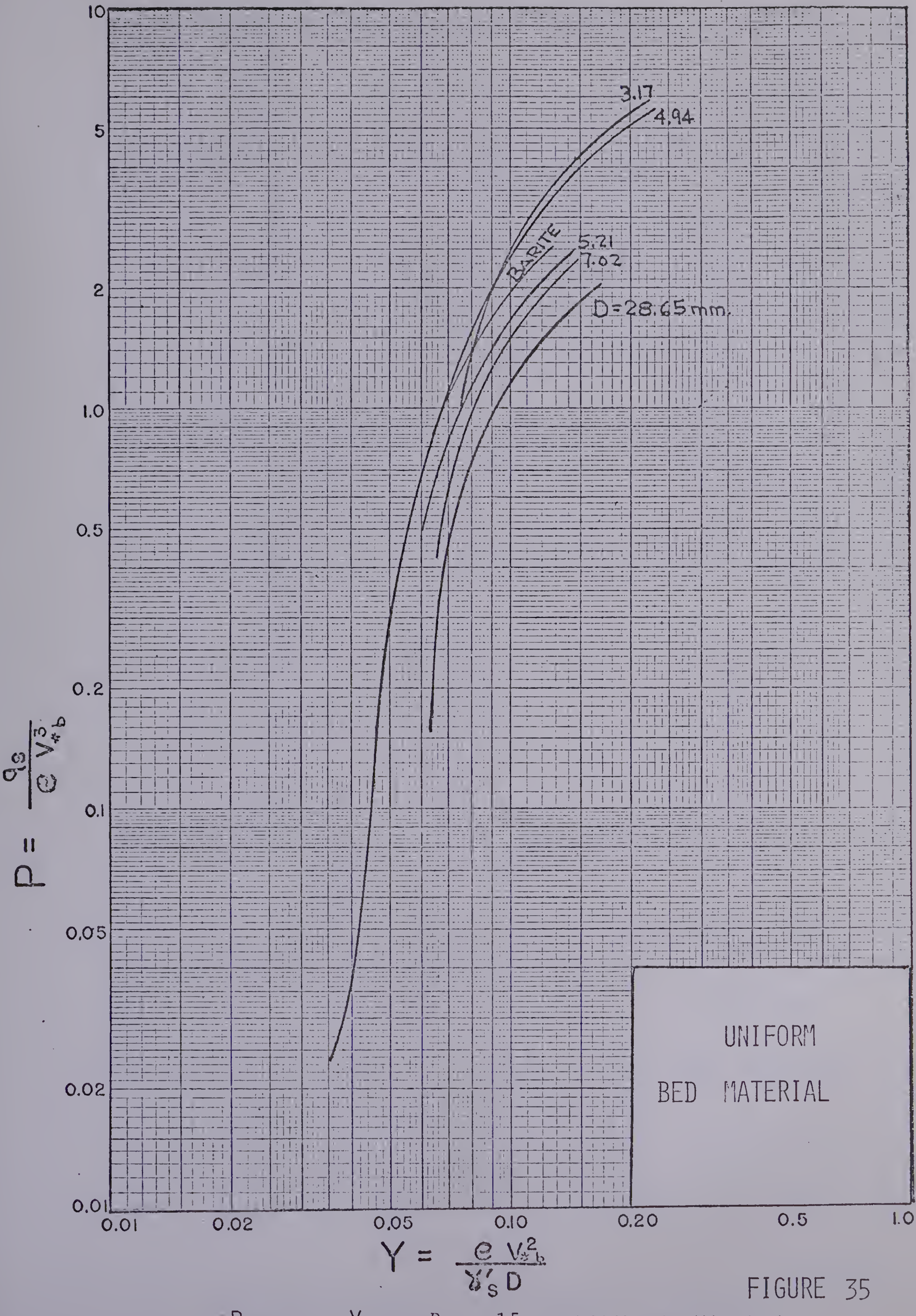


FIGURE 35

P VERSUS Y FOR $\frac{R_b}{D}$ = 15, UNIFORM BED MATERIAL

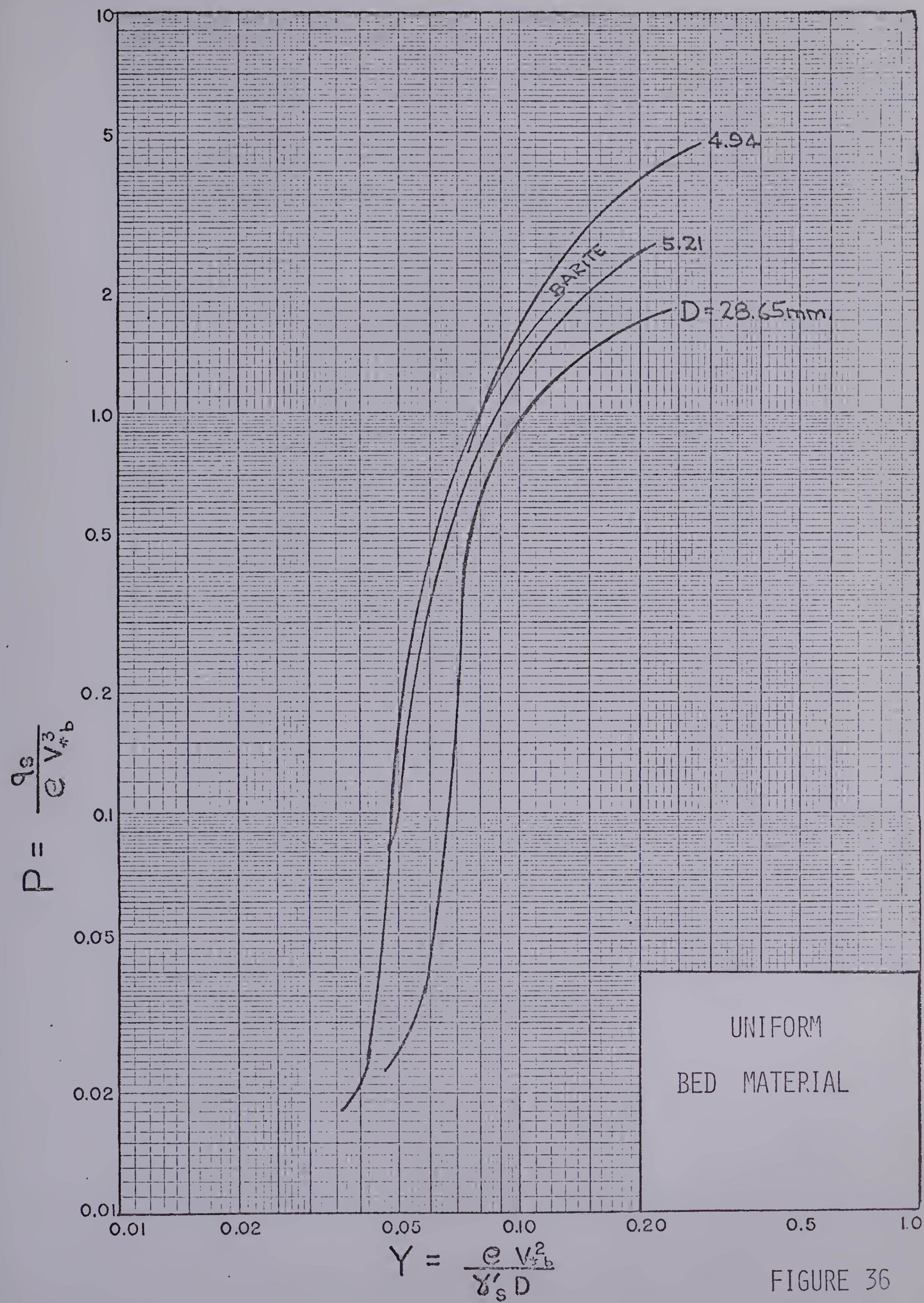


FIGURE 36

P VERSUS Y FOR $\frac{R_b}{D}$ = 20, UNIFORM BED MATERIAL

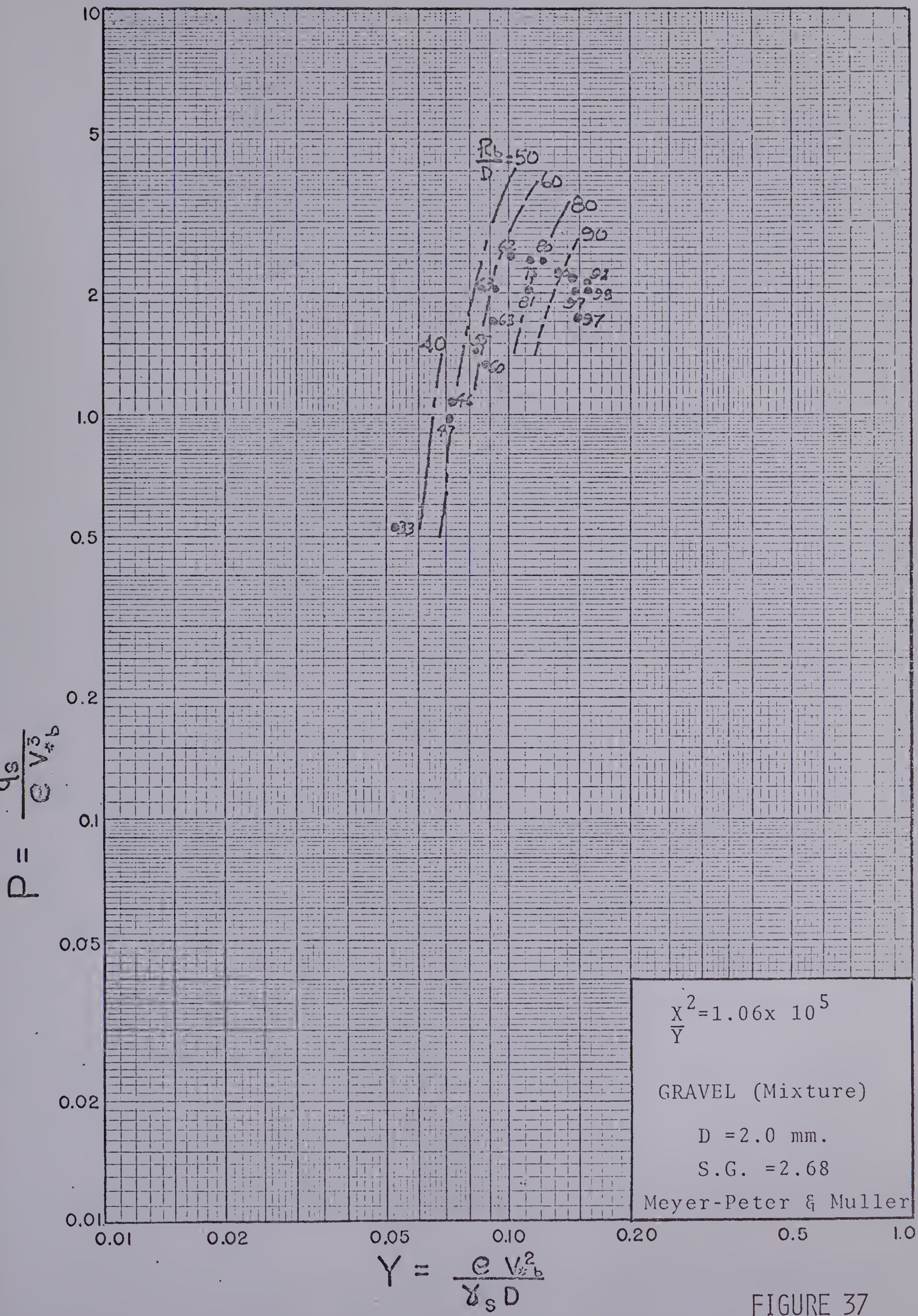


FIGURE 37

P VERSUS Y FOR D =2.0mm.-(Mixture)

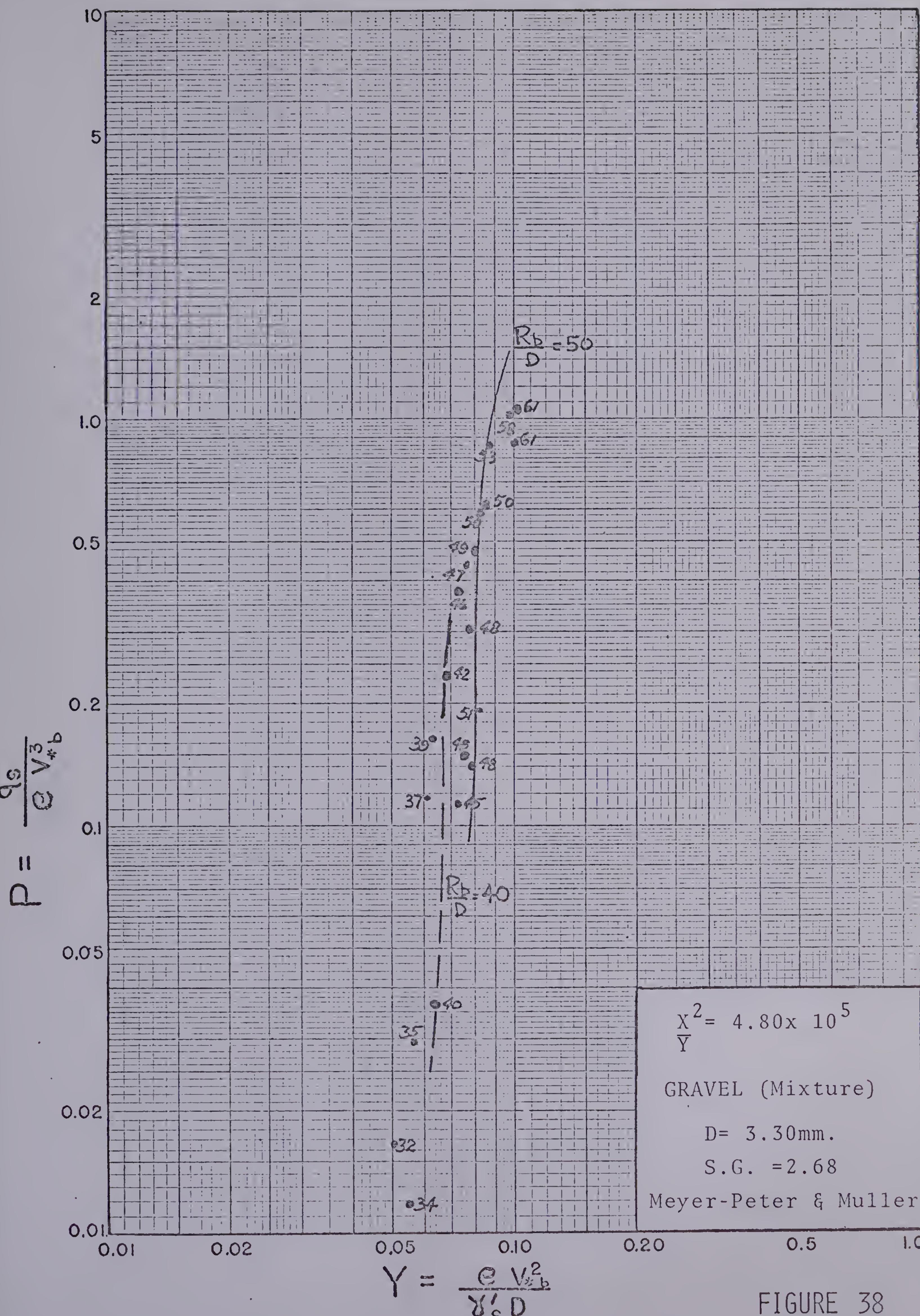


FIGURE 38

P VERSUS Y FOR D= 3.30mm.- (Mixture)

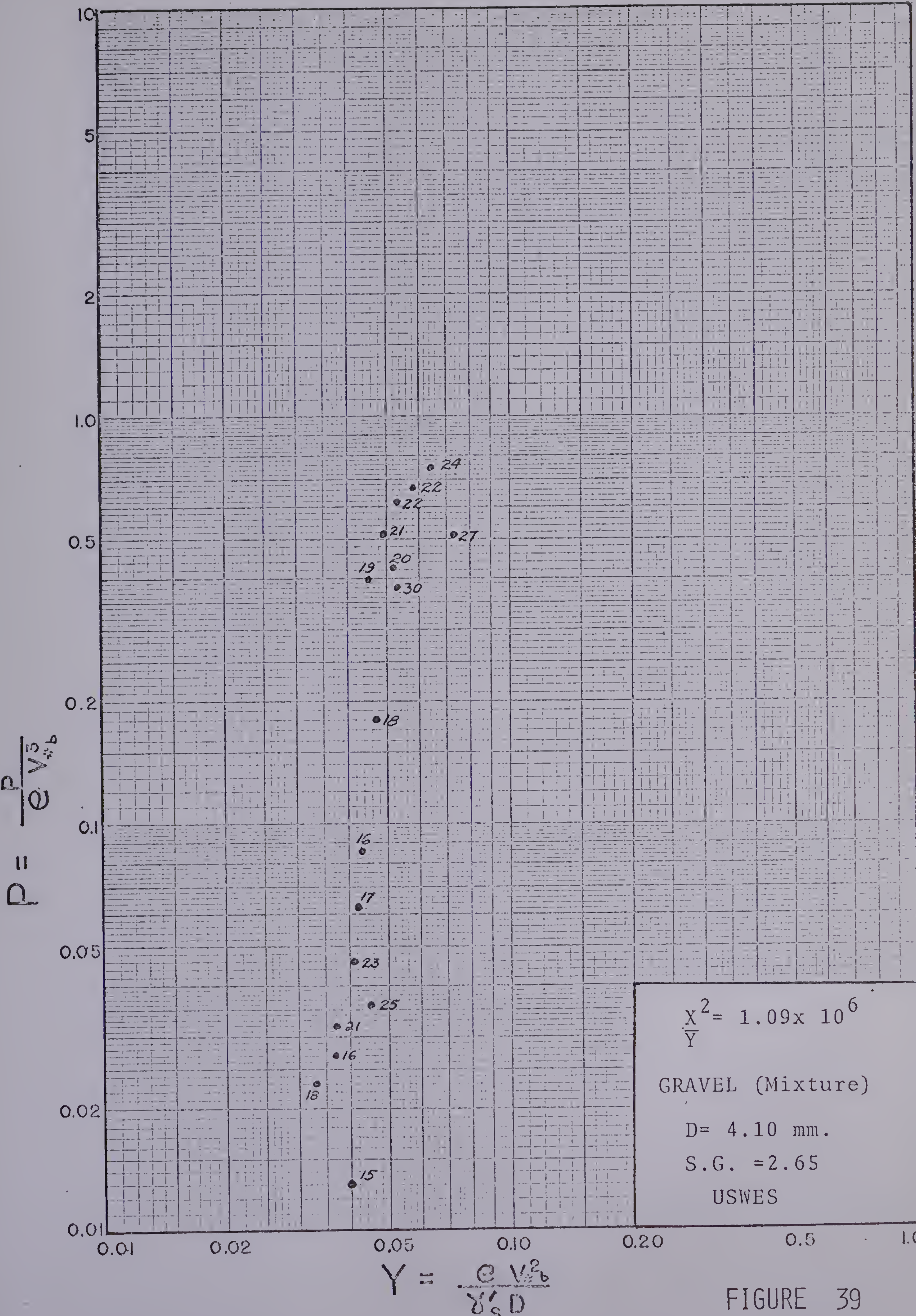


FIGURE 39

P VERSUS Y FOR D= 4.10 mm.- (Mixture)

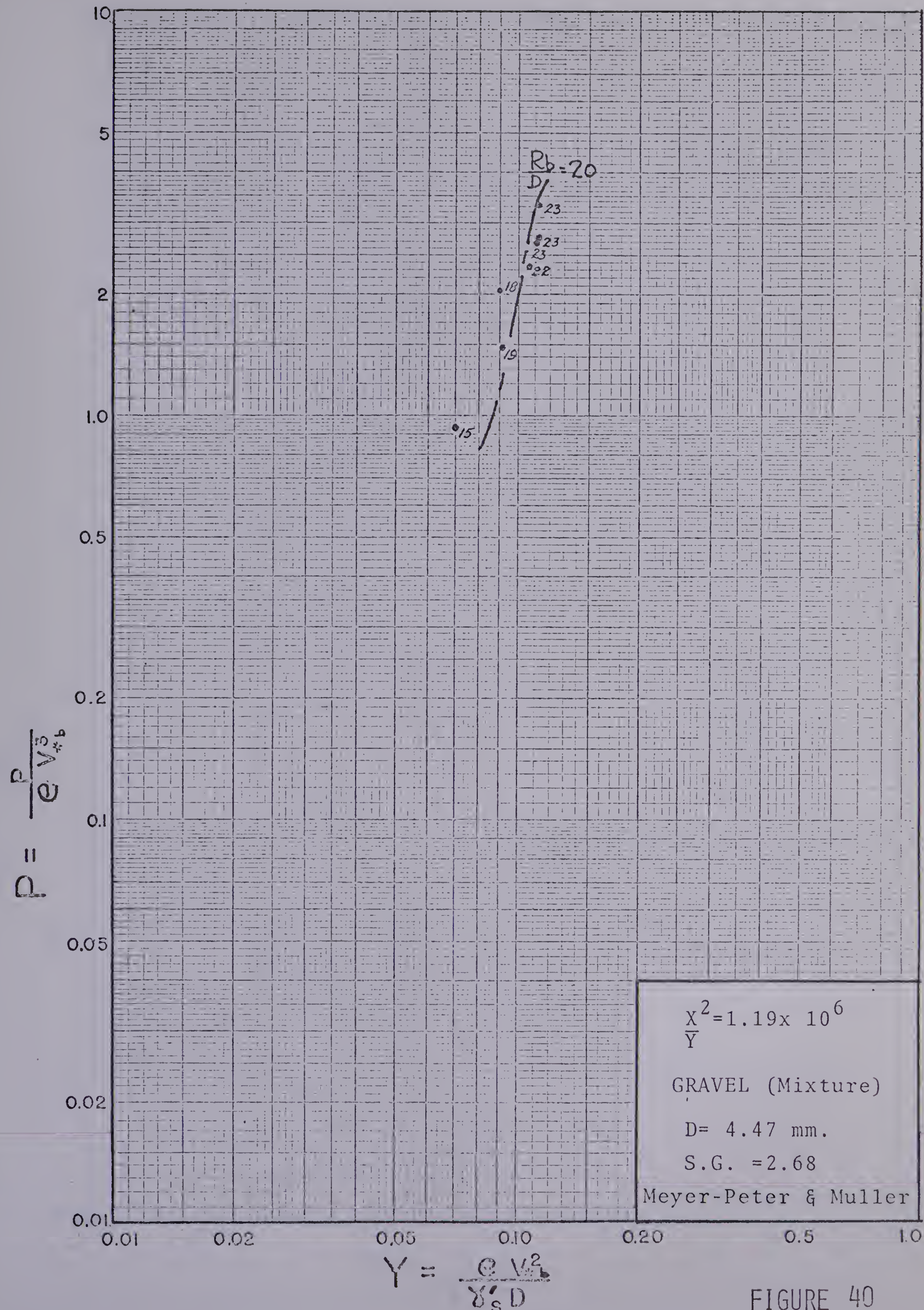


FIGURE 40

P VERSUS Y FOR D =4.47 mm.-(Mixture)

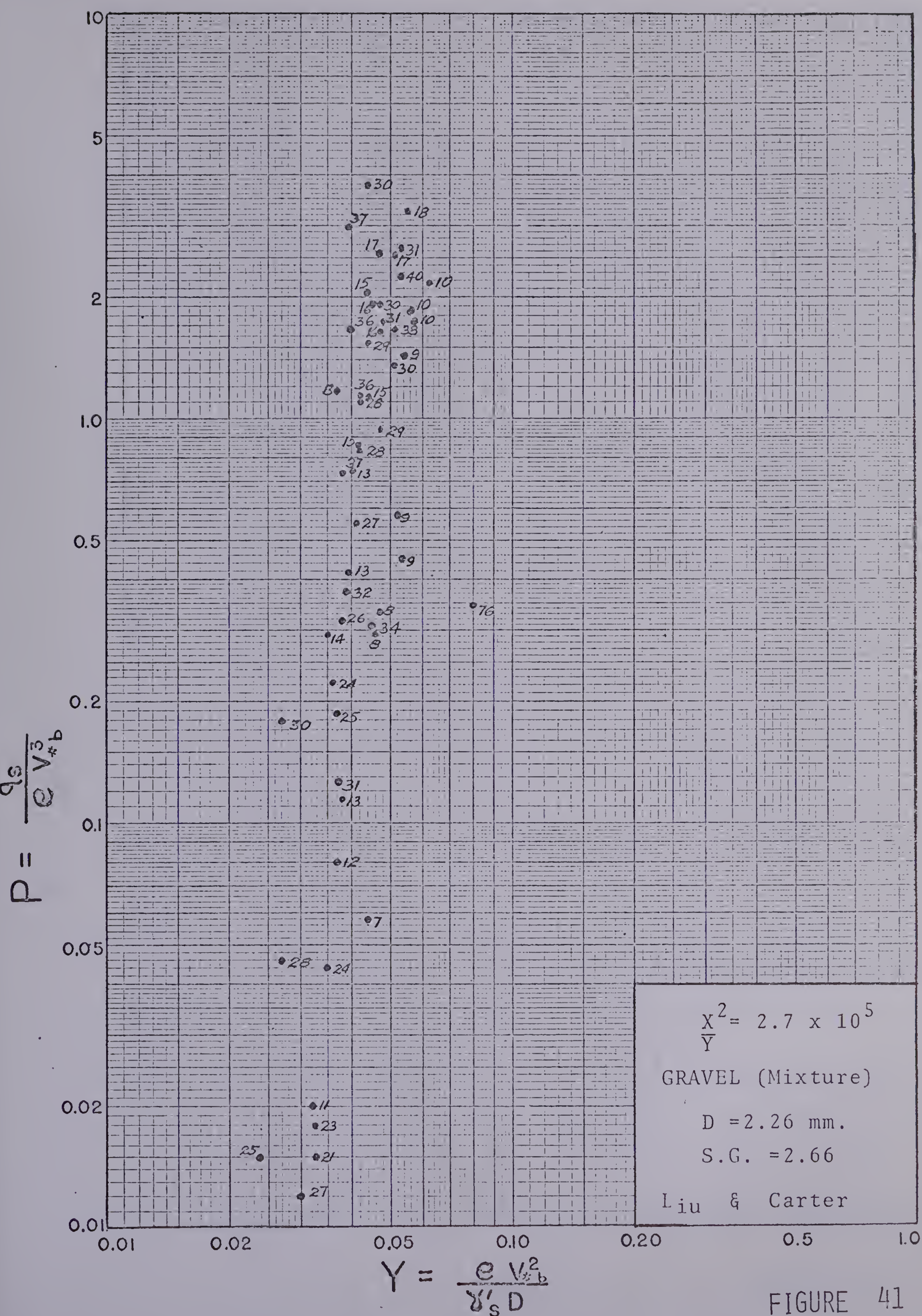


FIGURE 41

P VERSUS Y FOR D = 2.26 mm.-(Mixture)

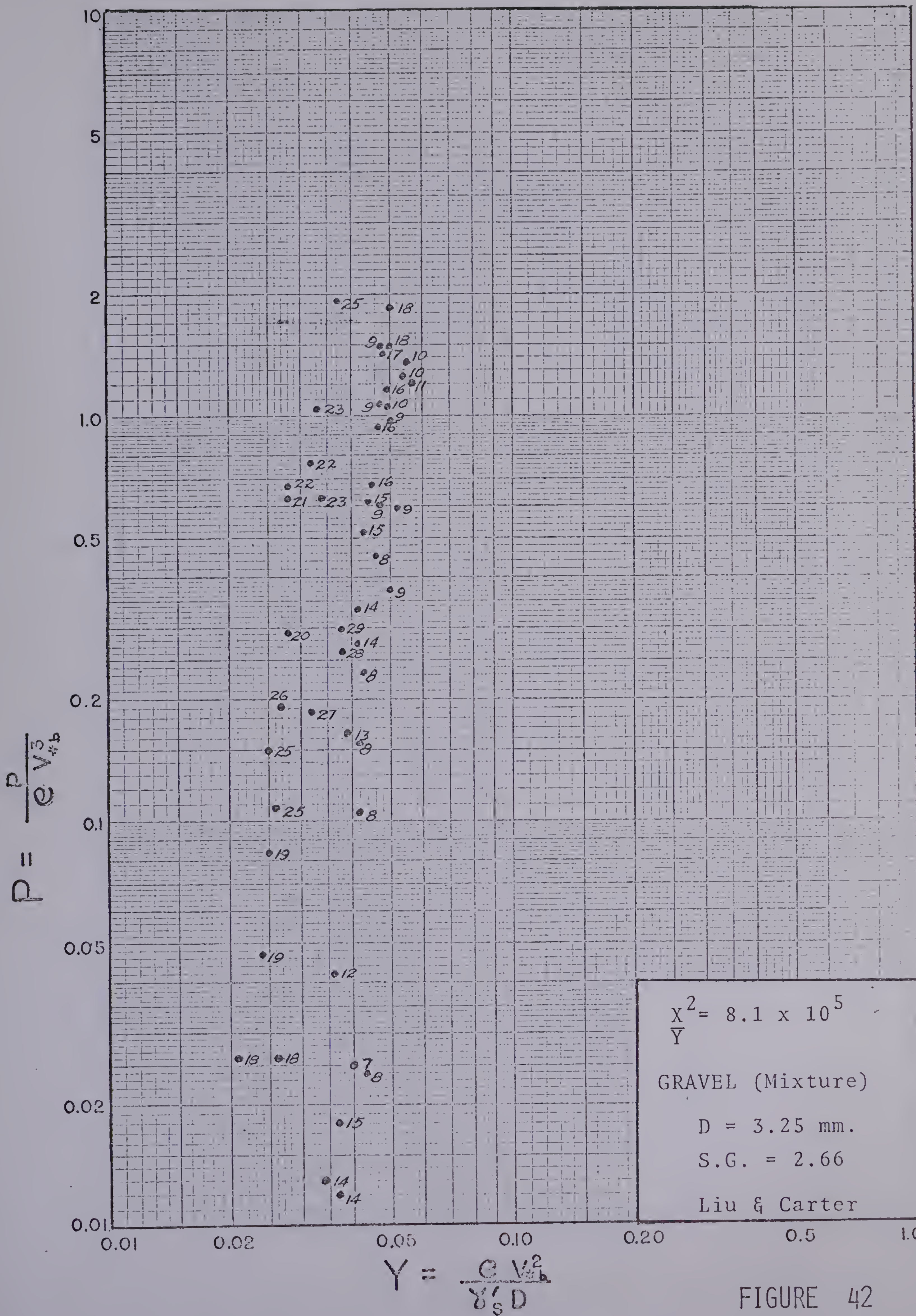


FIGURE 42

P VERSUS Y FOR D =3.25 mm.- (Mixture)

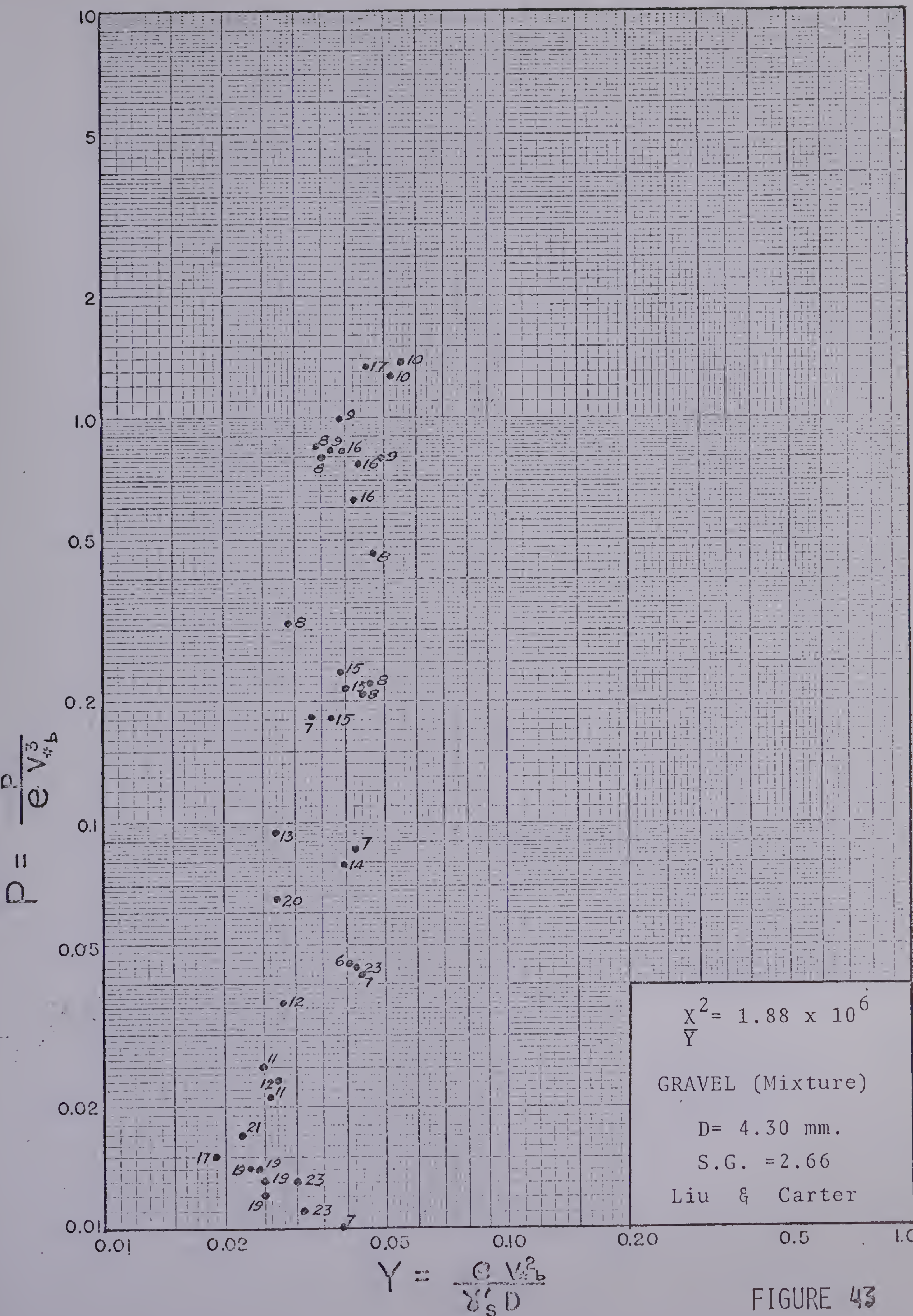


FIGURE 43

P VERSUS Y FOR D =4.30 mm.- (Mixture)

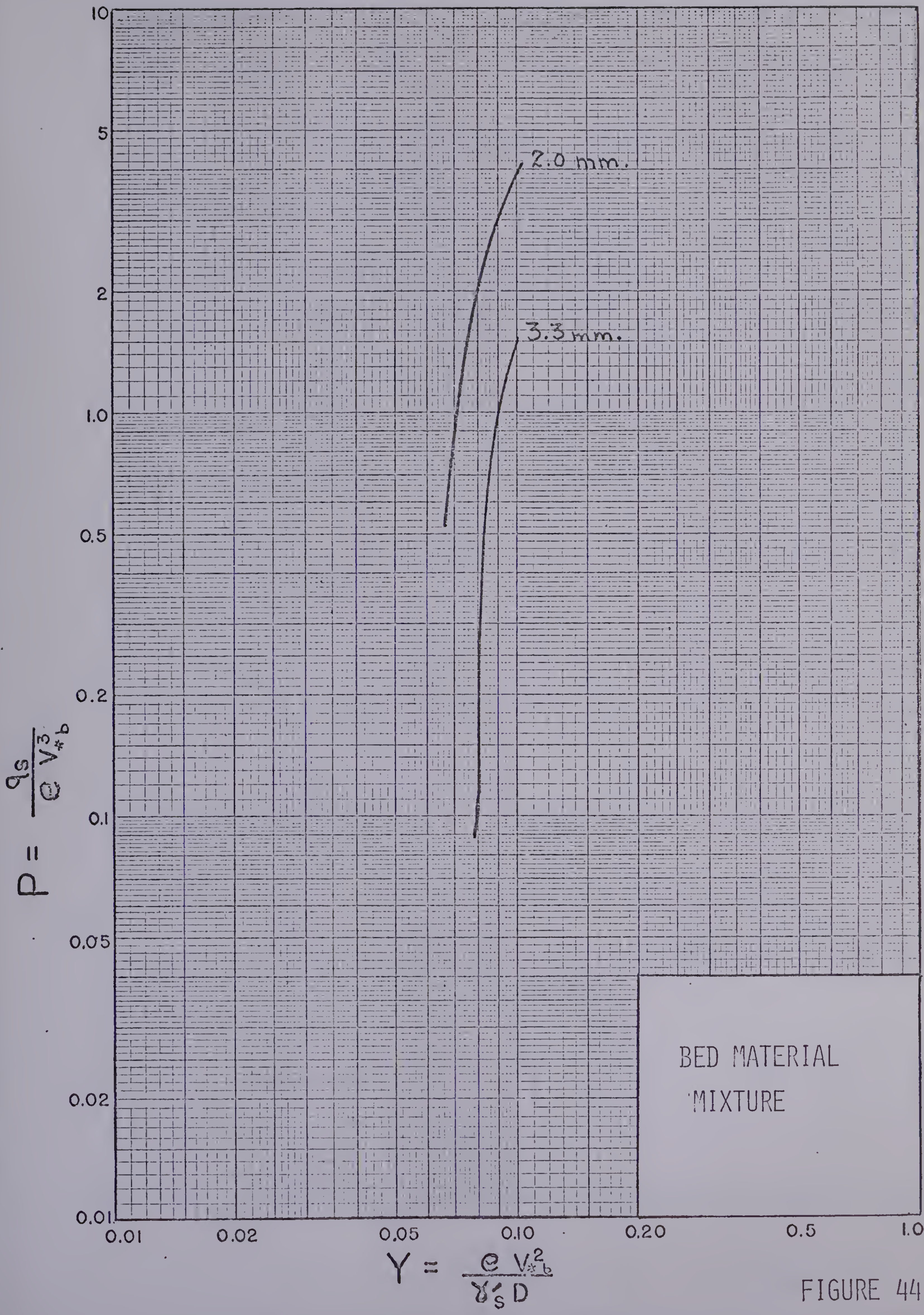


FIGURE 44

P VERSUS Y FOR $\frac{R_b}{D} = 50$, BED MATERIAL MIXTURE

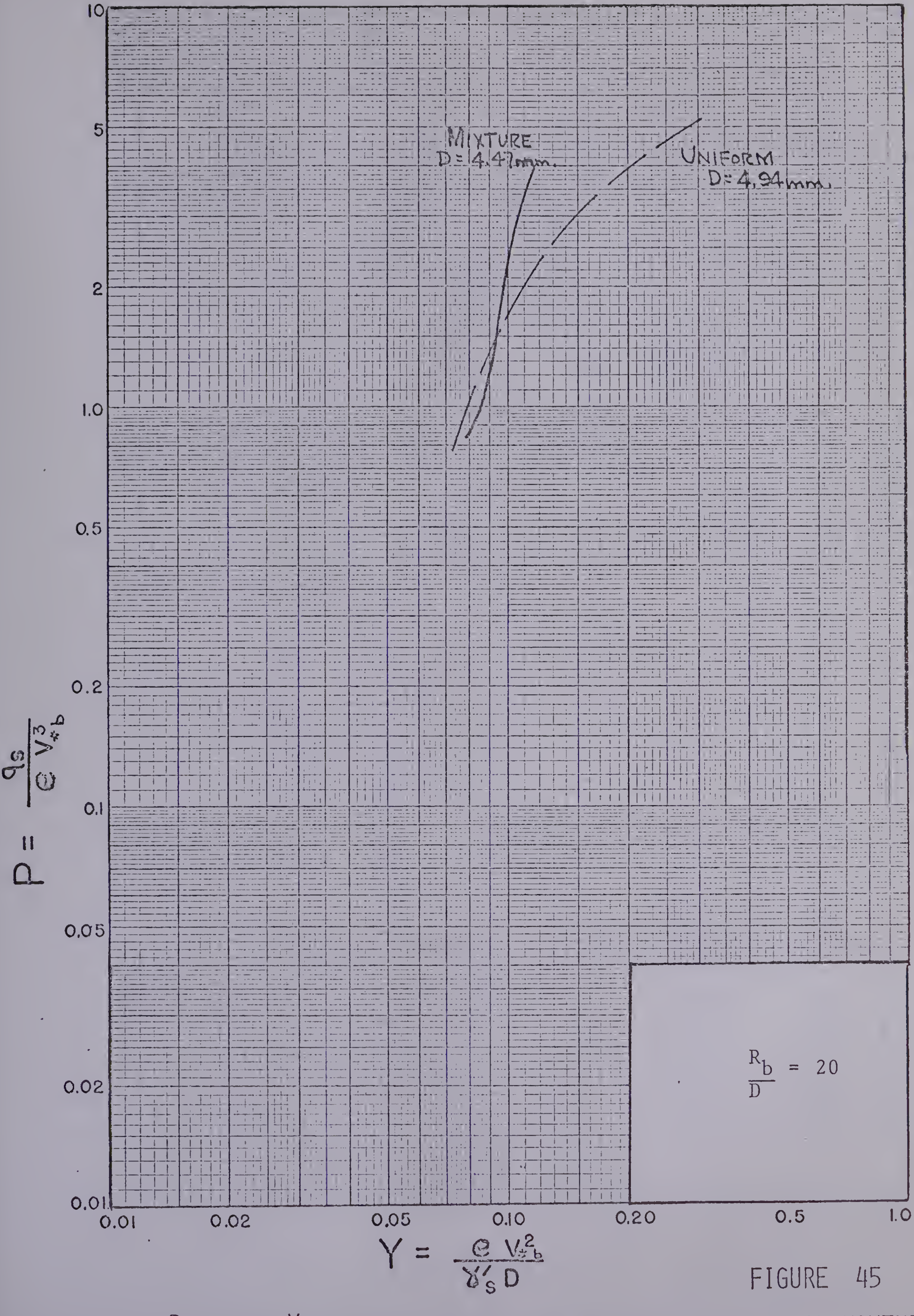


FIGURE 45

P VERSUS Y COMPARISON FOR UNIFORM AND BED MATERIAL MIXTURES

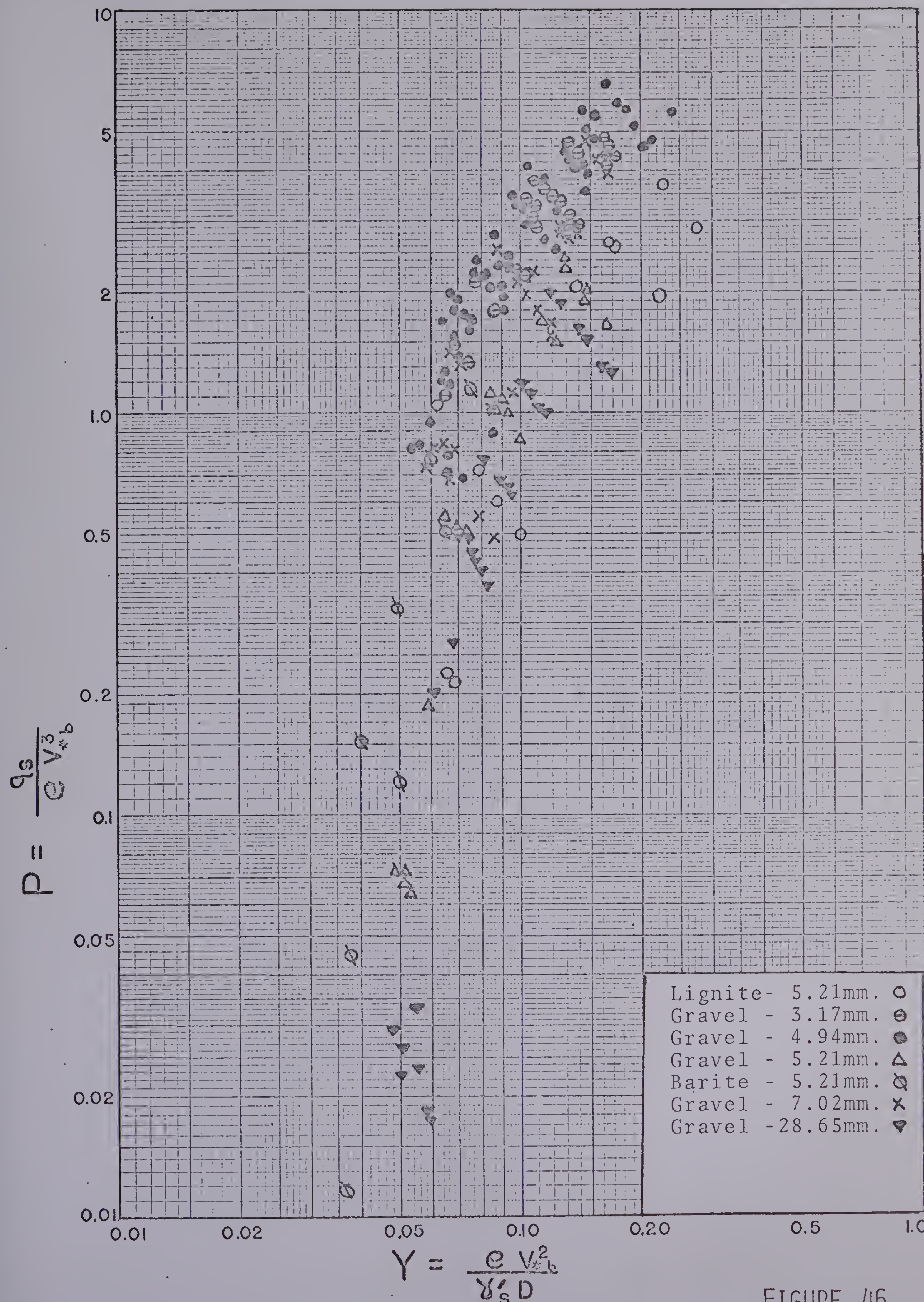
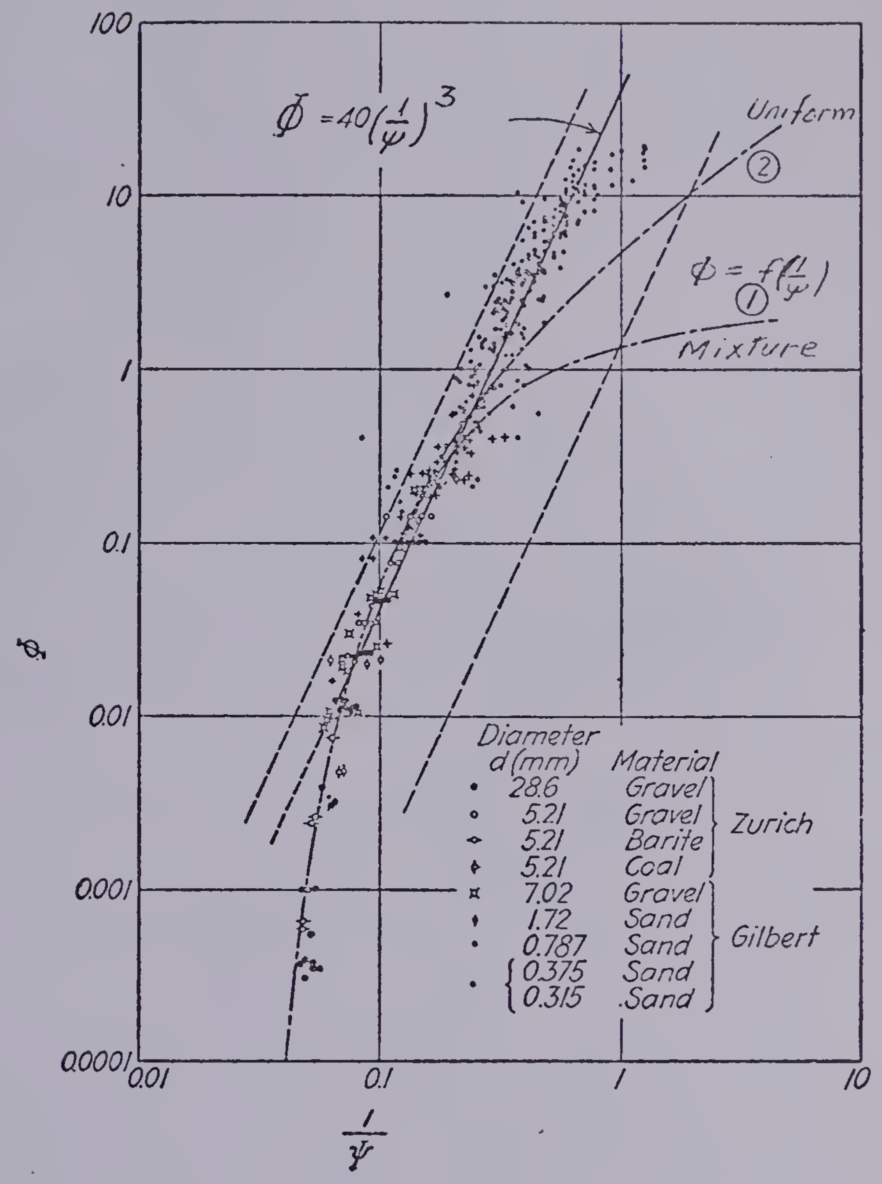


FIGURE 46

P VERSUS Y FOR ALL UNIFORM MATERIAL DATA



PLOT OF THE EINSTEIN BED - LOAD FUNCTION
(AFTER BROWN)

FIGURE 47

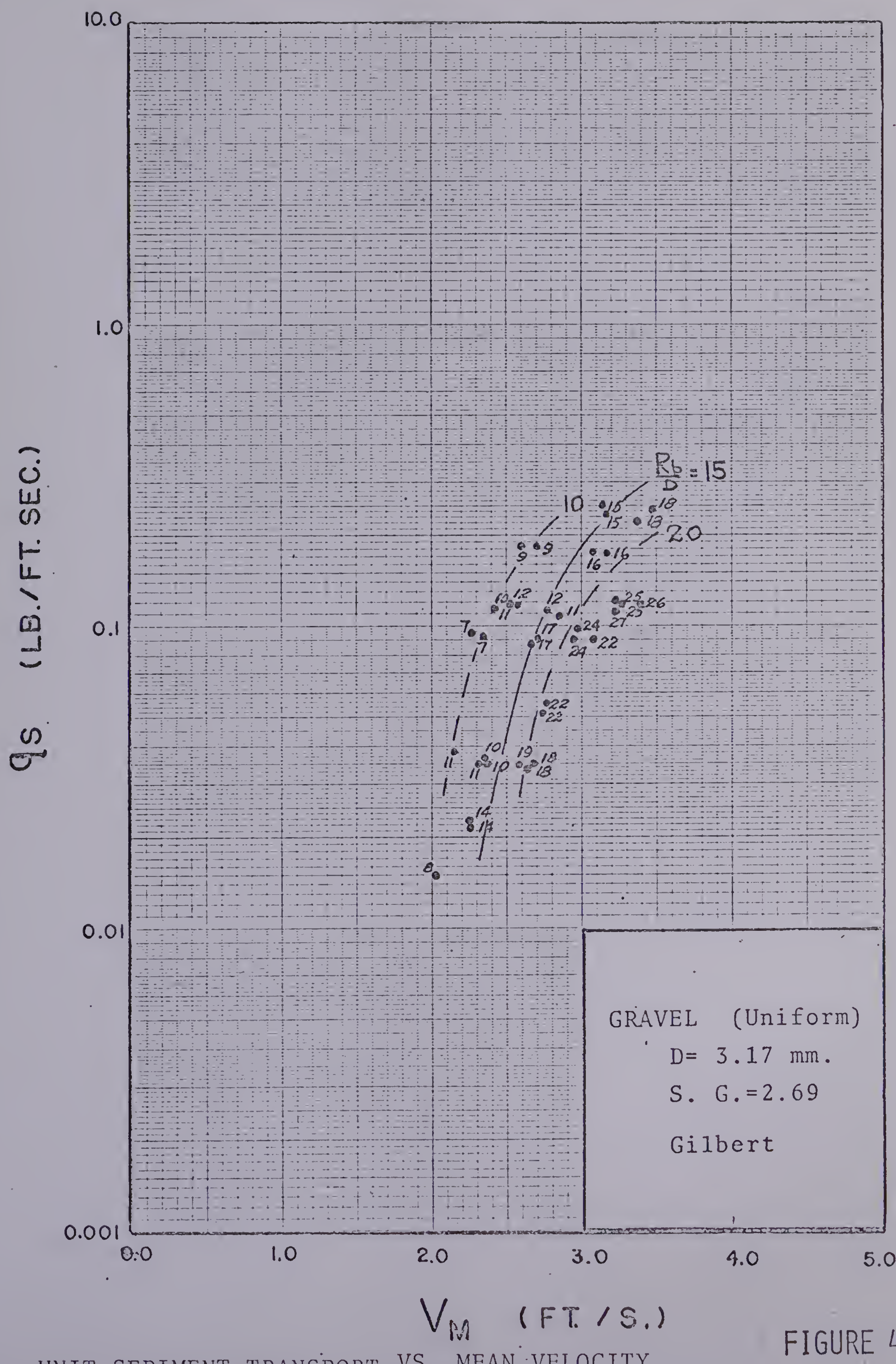


FIGURE 48

UNIT SEDIMENT TRANSPORT VS. MEAN VELOCITY

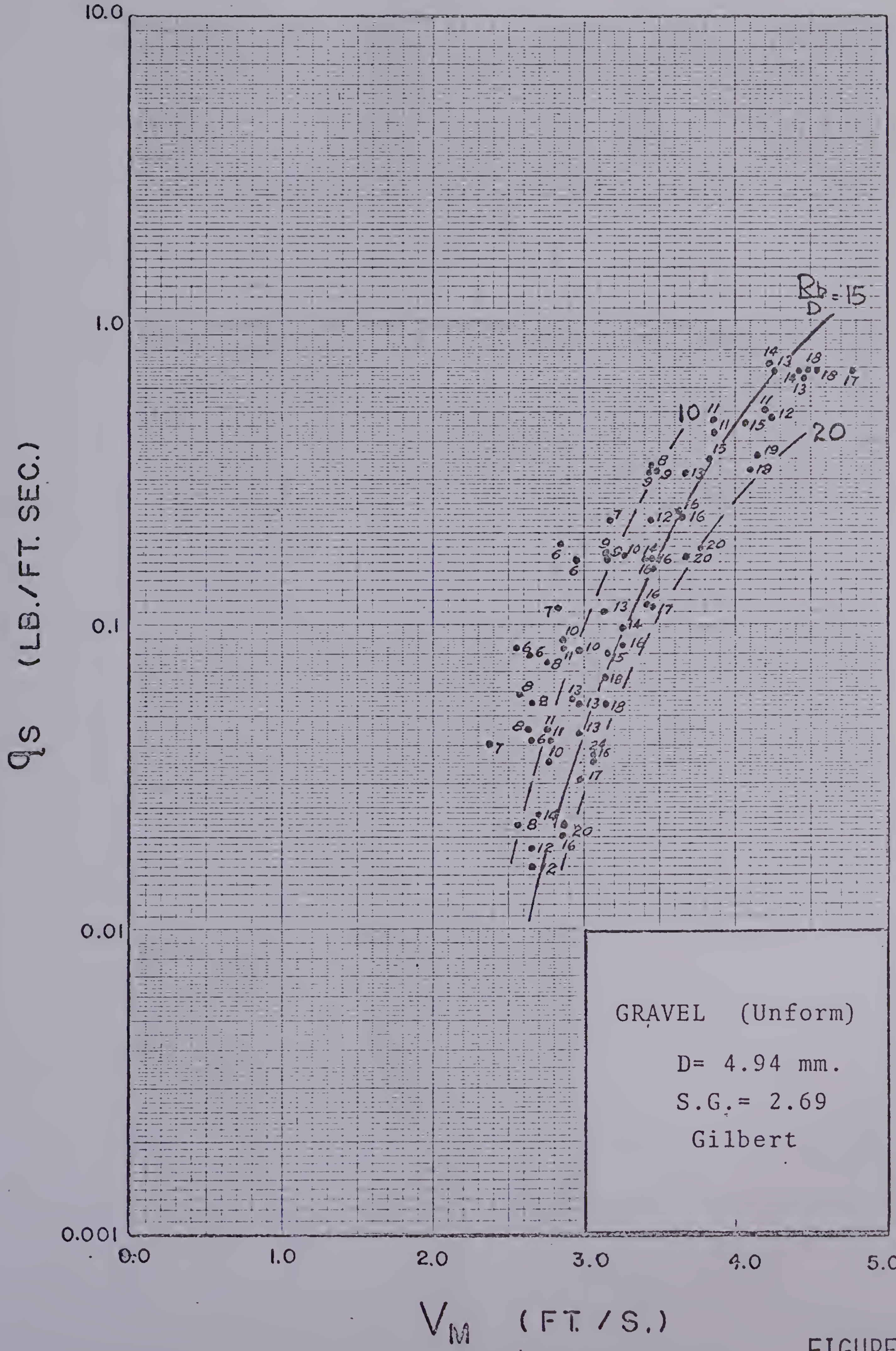


FIGURE 49

UNIT SEDIMENT TRANSPORT VS. MEAN VELOCITY

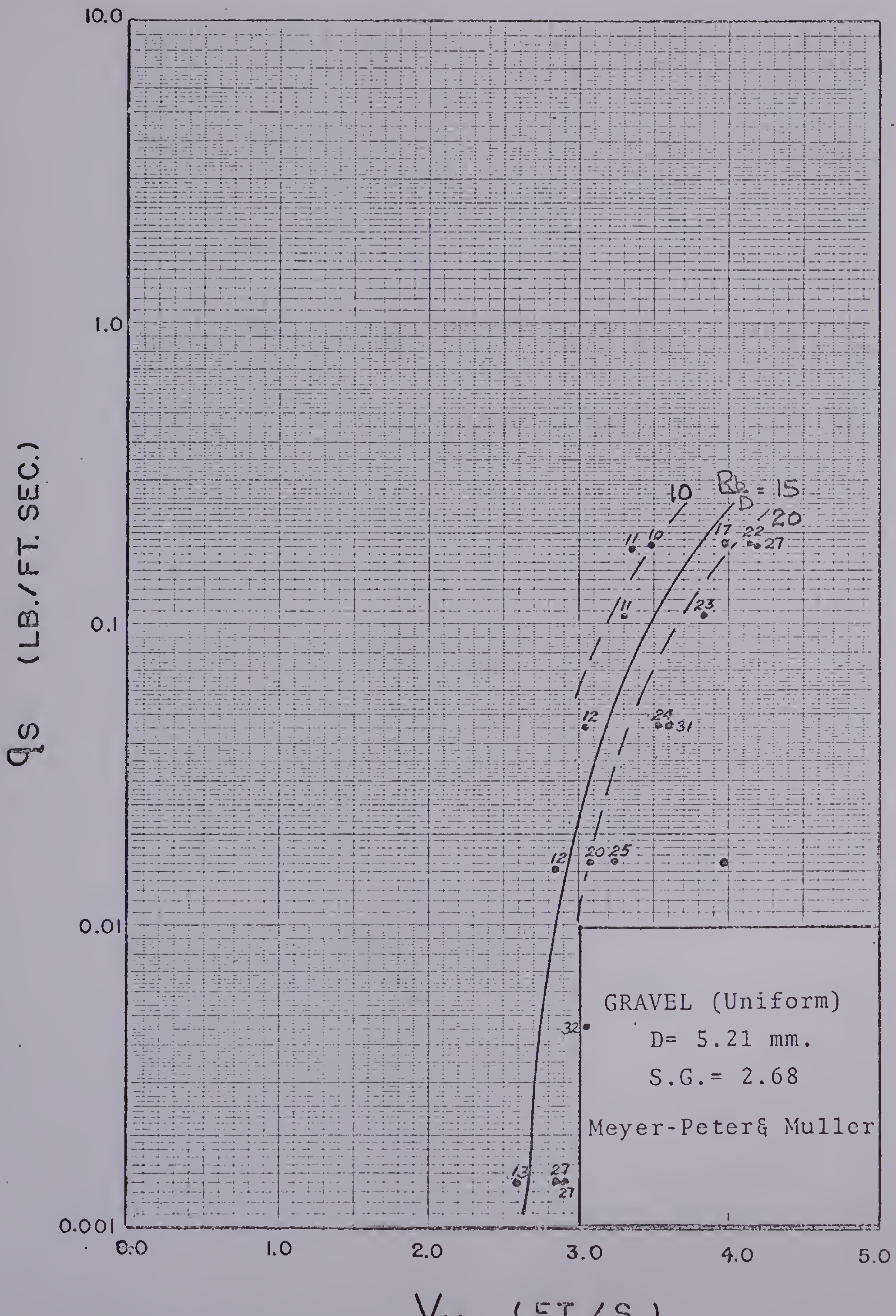


FIGURE 50

UNIT SEDIMENT TRANSPORT VS. MEAN VELOCITY

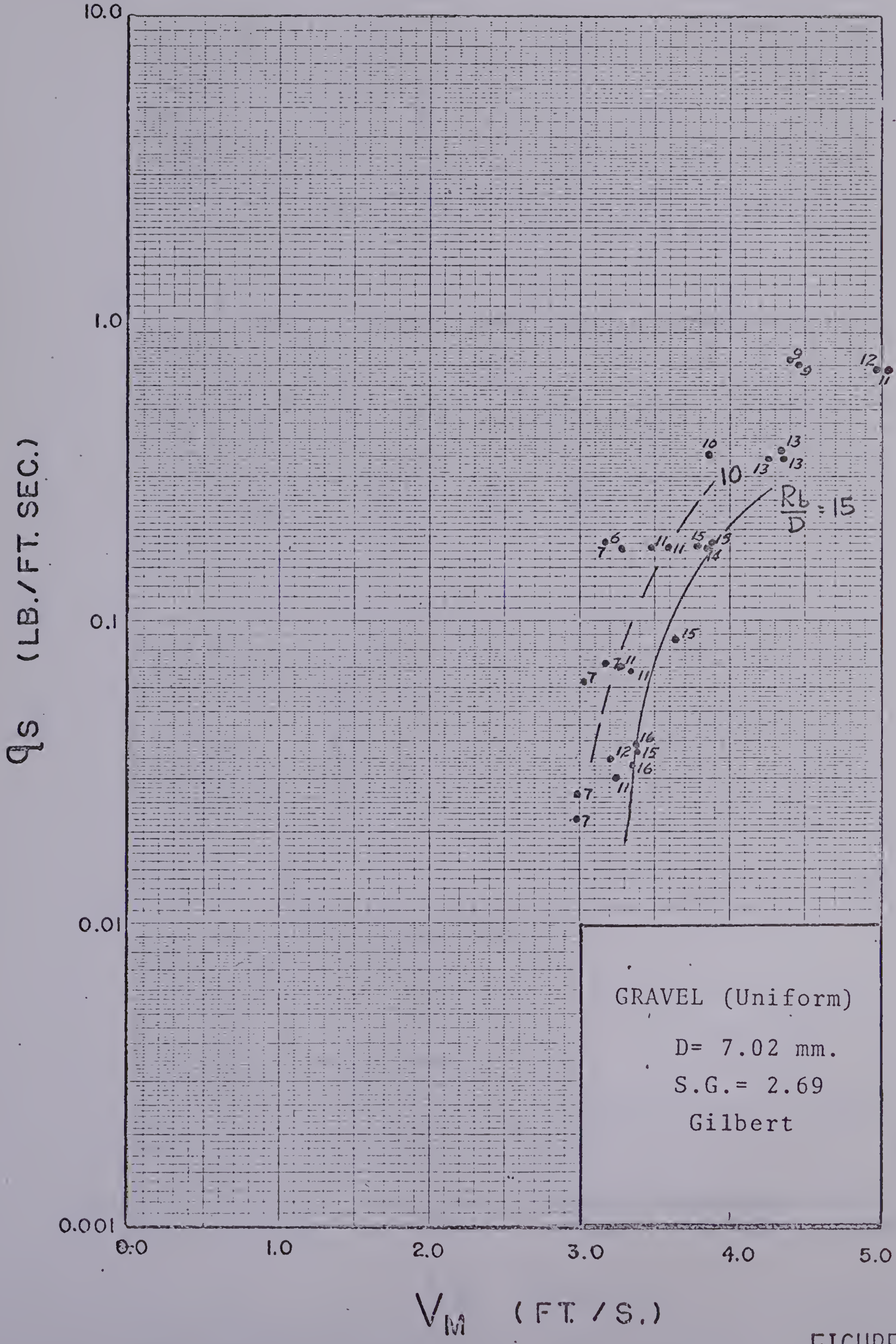


FIGURE 51

UNIT SEDIMENT TRANSPORT VS. MEAN VELOCITY

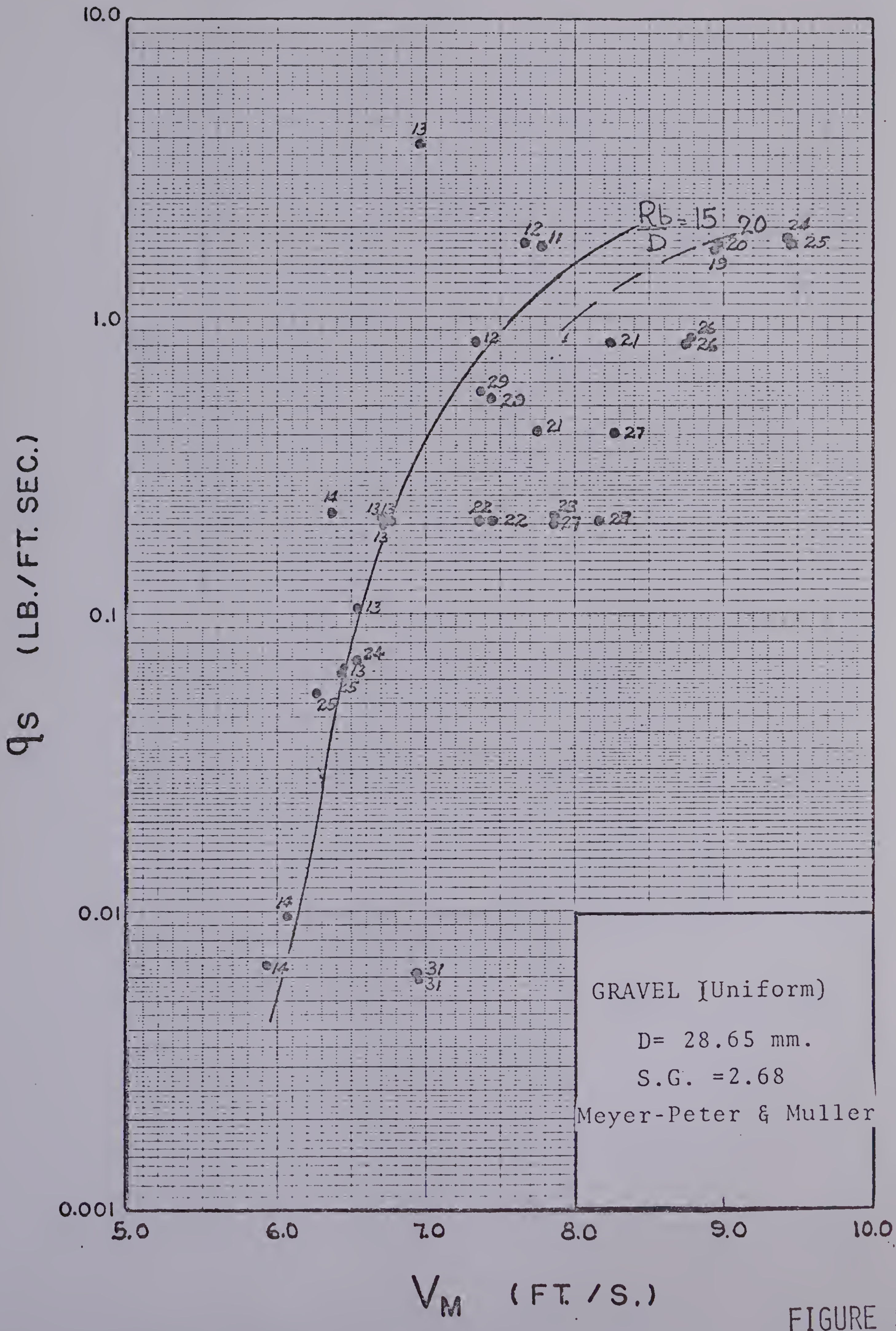


FIGURE 52

UNIT SEDIMENT TRANSPORT VS. MEAN VELOCITY

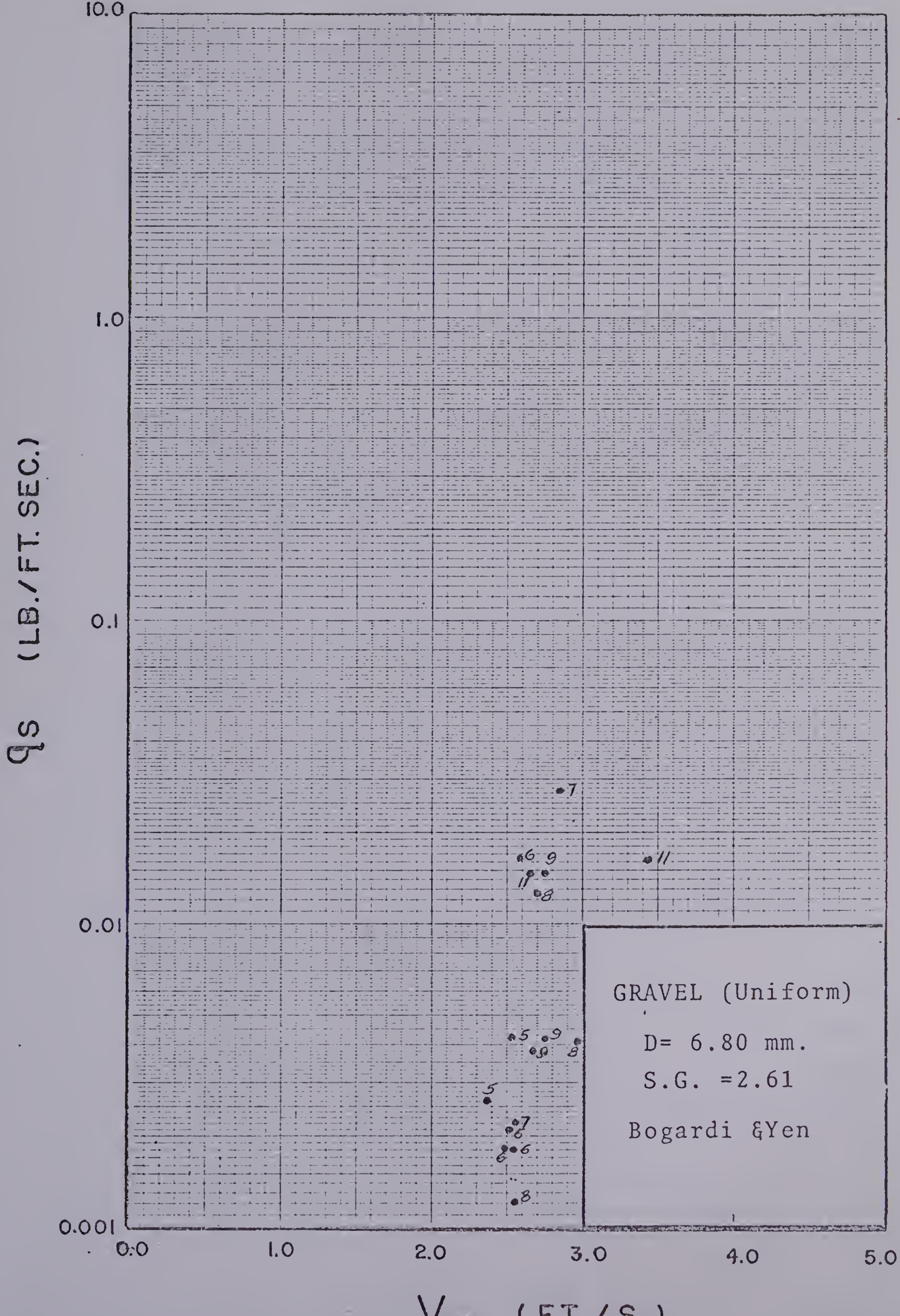


FIGURE 53

UNIT SEDIMENT TRANSPORT VS. MEAN VELOCITY

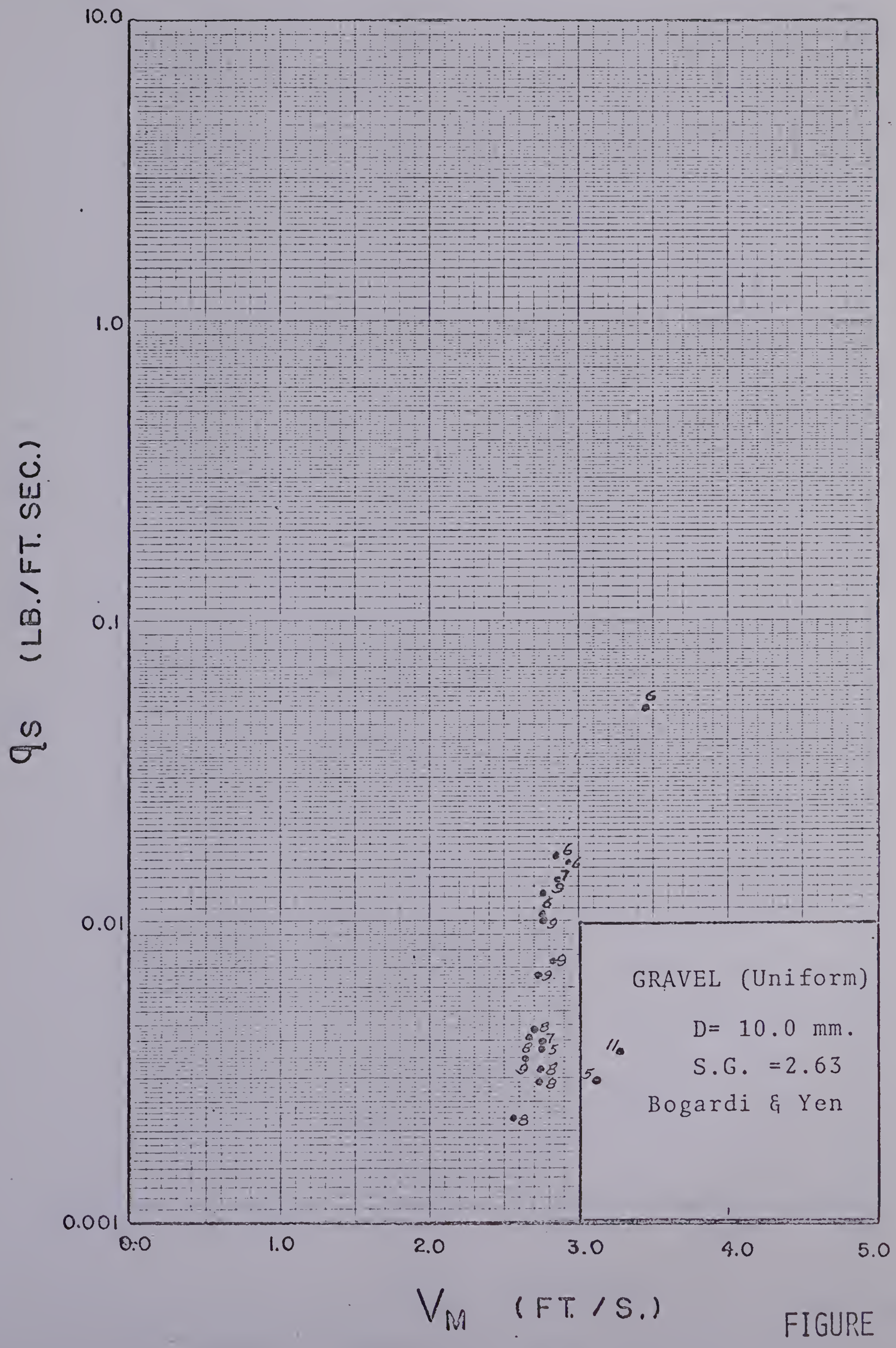


FIGURE 54

UNIT SEDIMENT TRANSPORT VS. MEAN VELOCITY

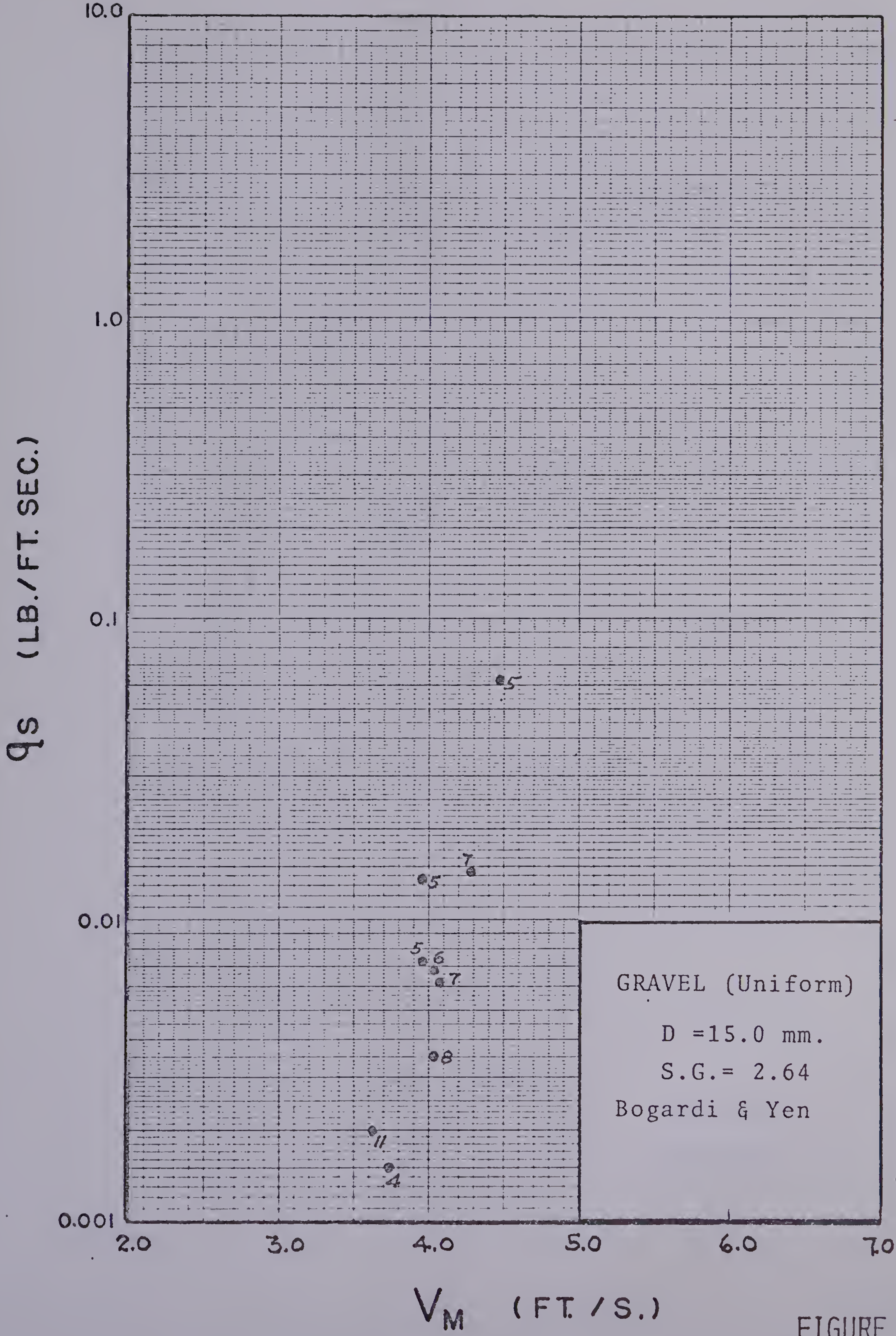


FIGURE 55

UNIT SEDIMENT TRANSPORT VS. MEAN VELOCITY

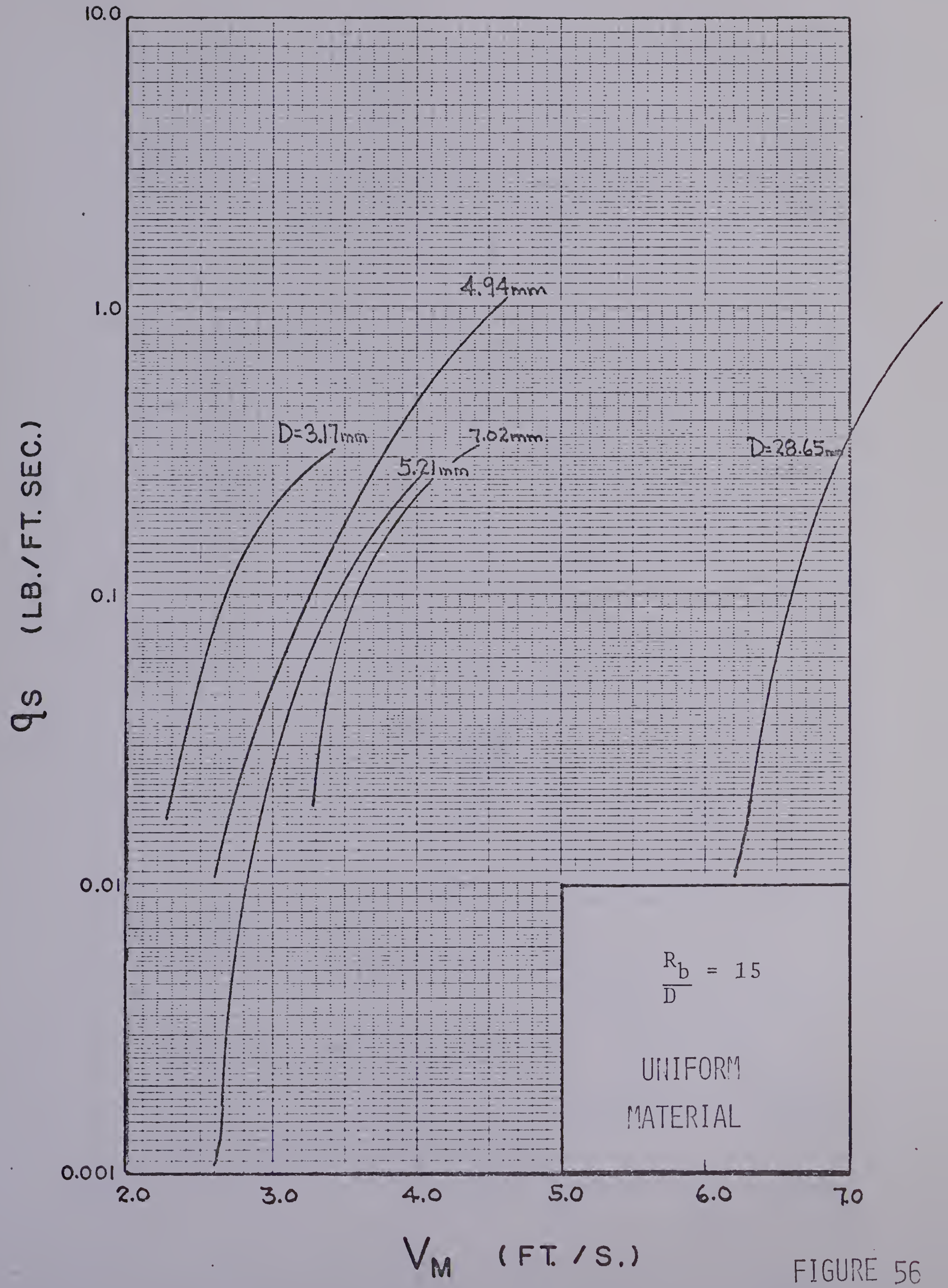
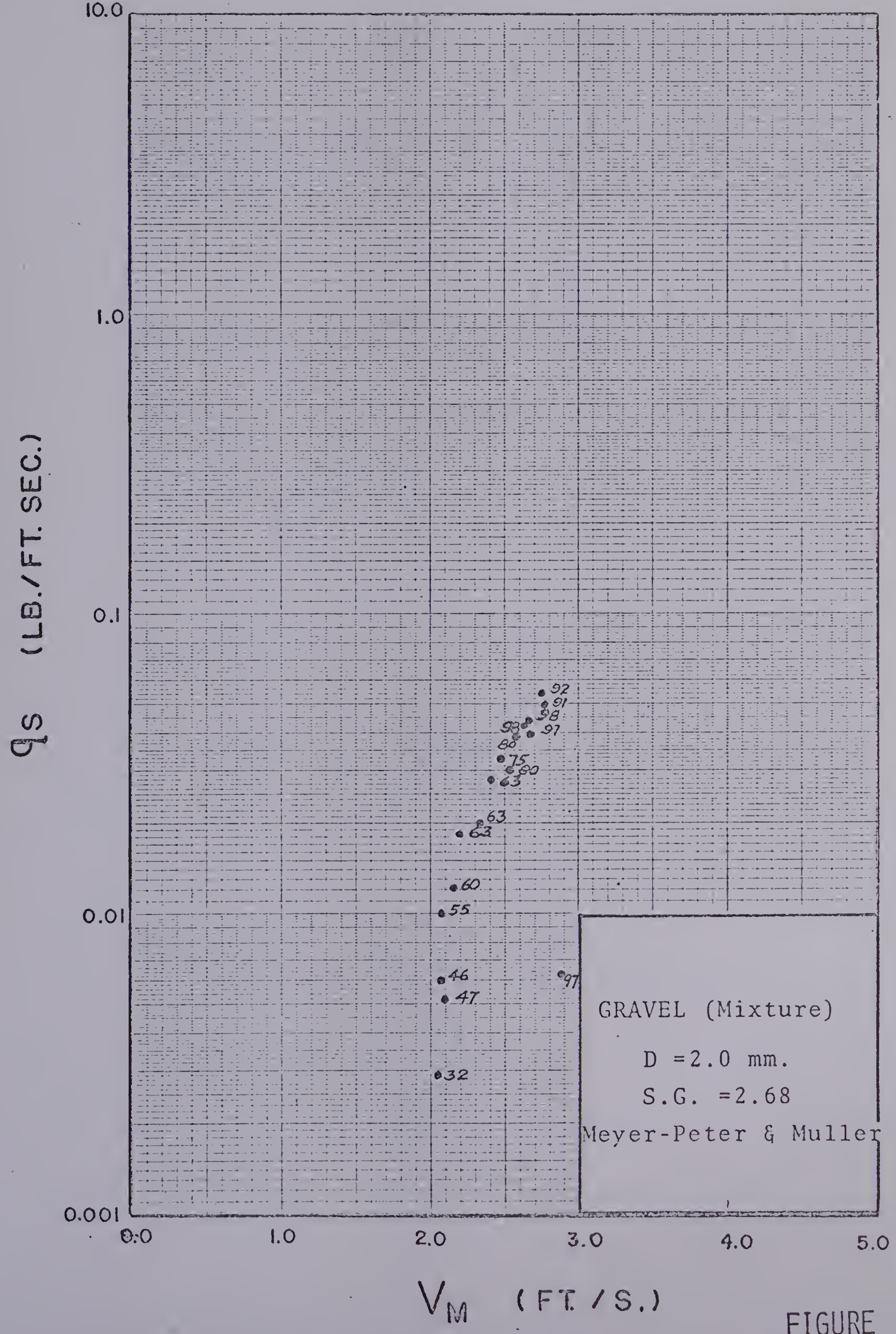


FIGURE 56

UNIT SEDIMENT TRANSPORT VERSUS MEAN VELOCITY



10.0
1.0
0.1
0.01
0.001
0.0
1.0
2.0
3.0
4.0
5.0
qs (LB./FT. SEC.)
VM (FT./S.)
92
91
98
93
97
88
75
80
63
63
63
60
55
46
47
97
32
GRAVEL (Mixture)
D =2.0 mm.
S.G. =2.68
Meyer-Peter & Muller

FIGURE 57

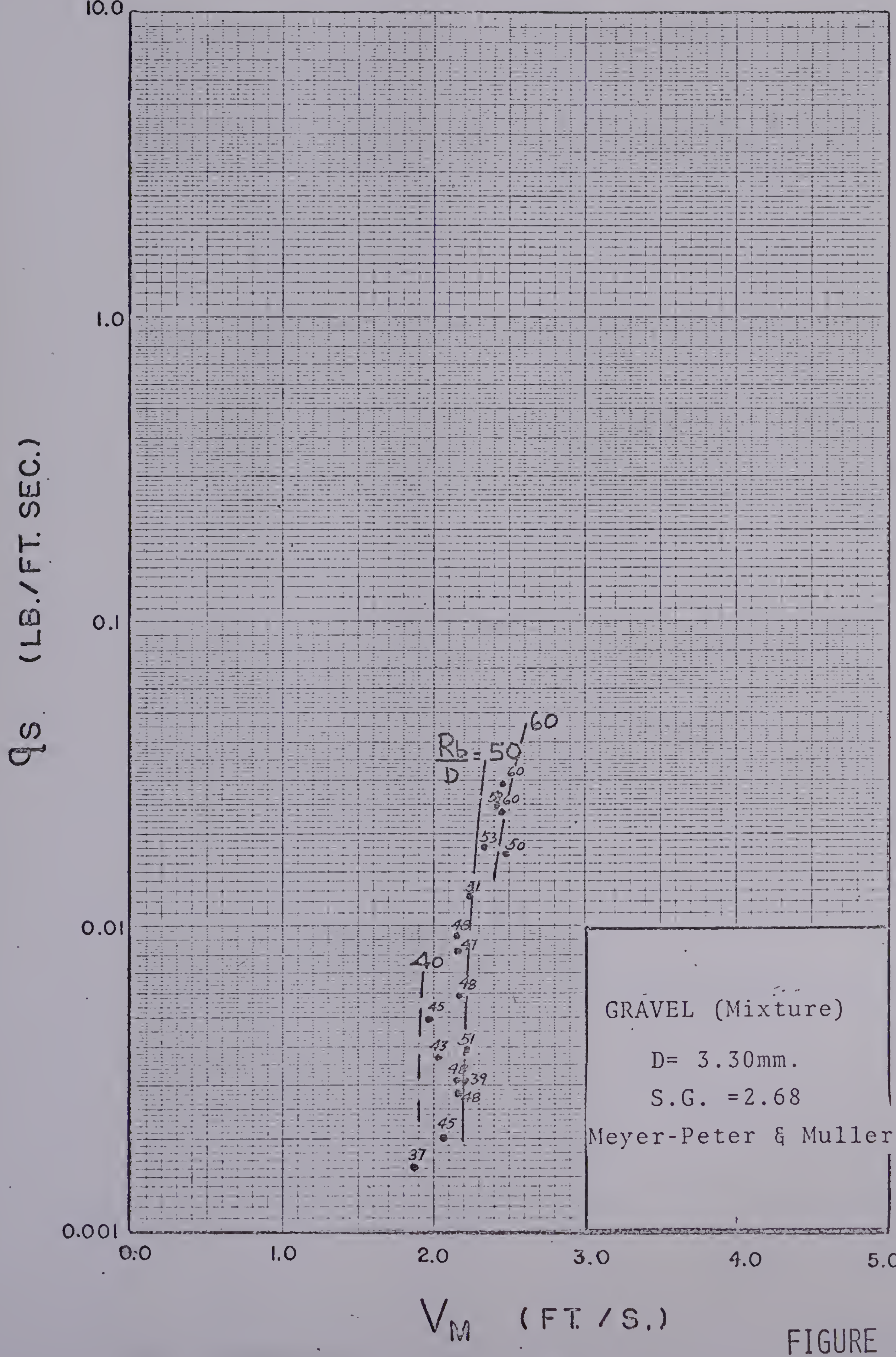
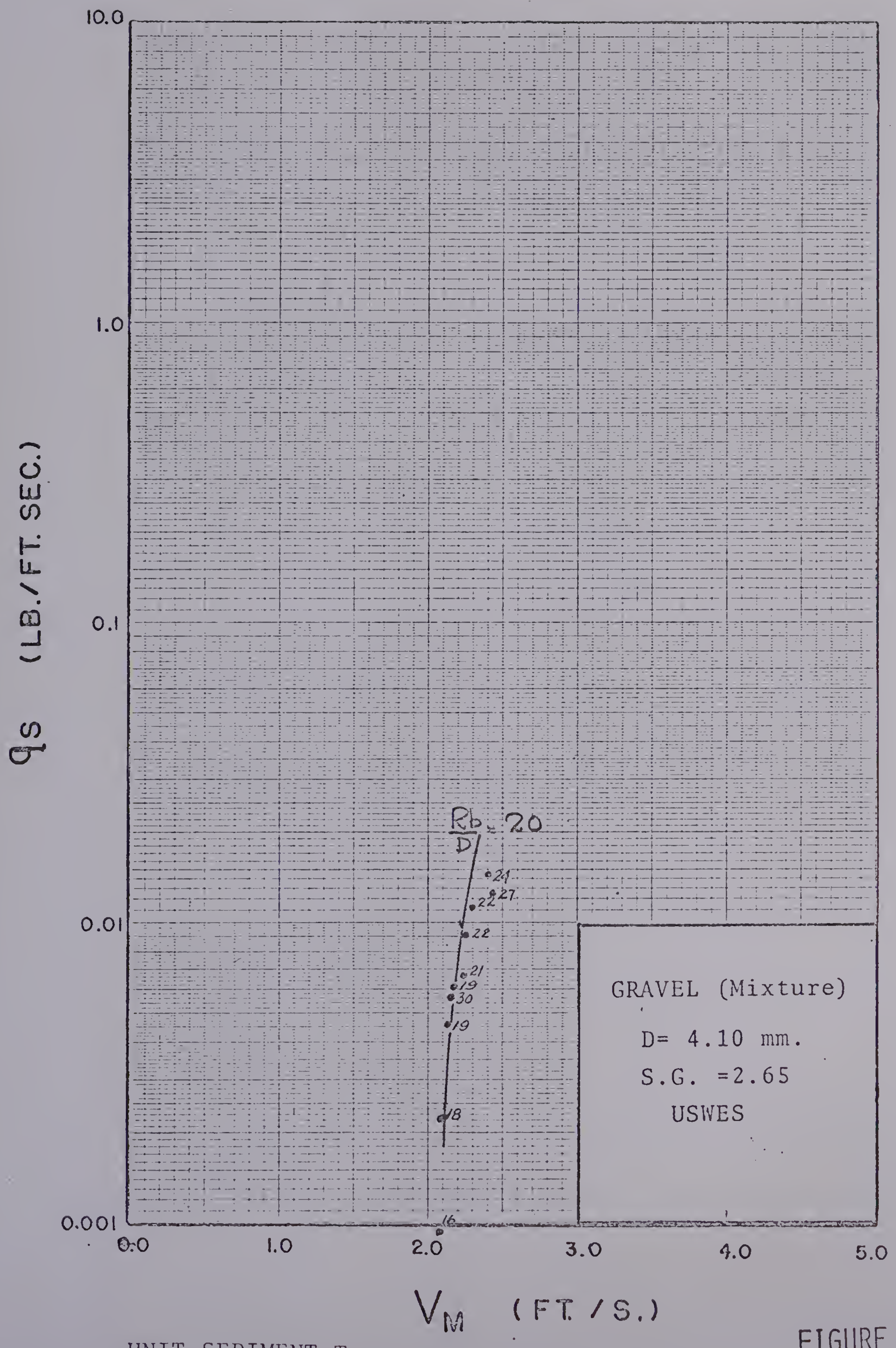


FIGURE 58

UNIT SEDIMENT TRANSPORT VS. MEAN VELOCITY



UNIT SEDIMENT TRANSPORT VS. MEAN VELOCITY

FIGURE 59

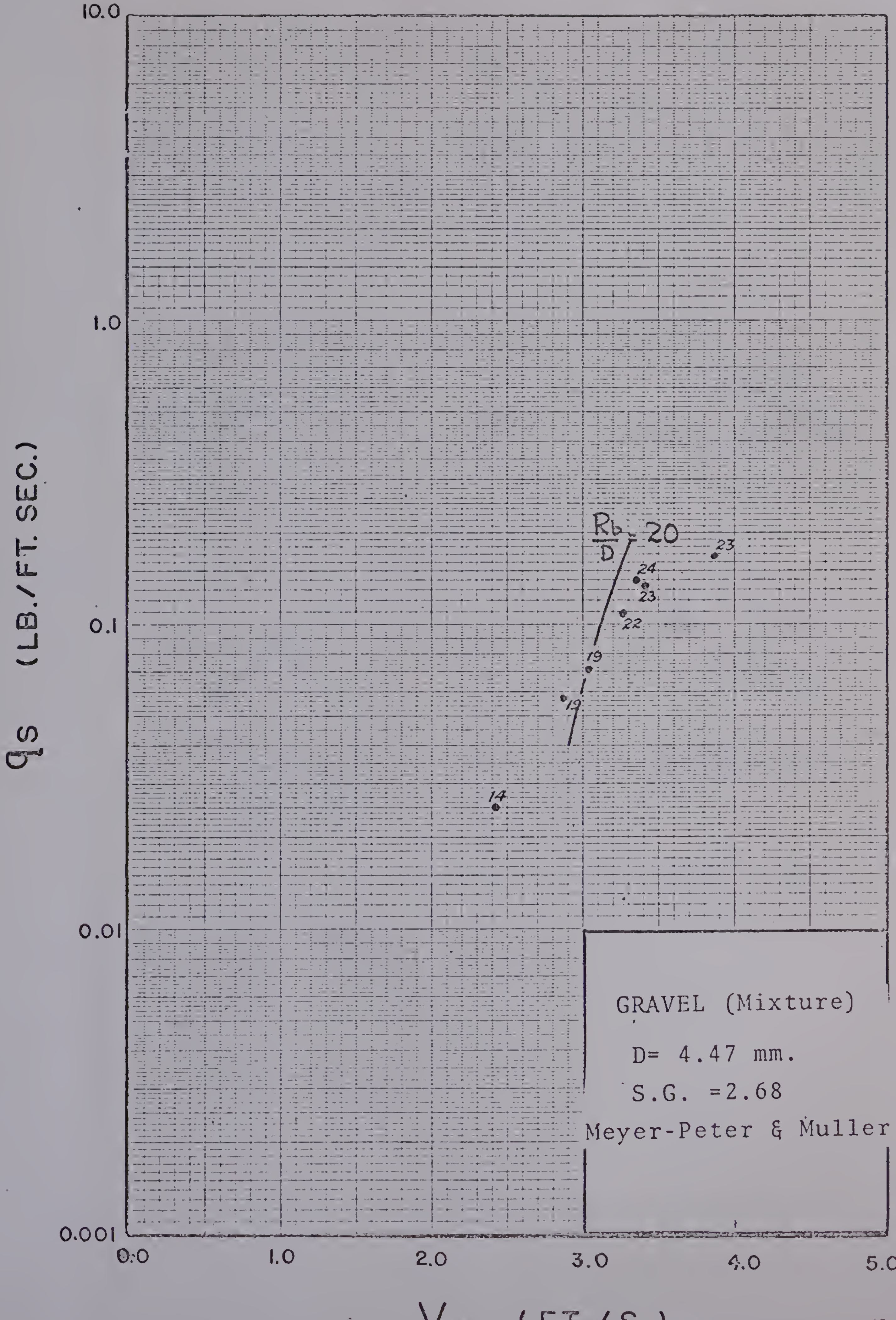
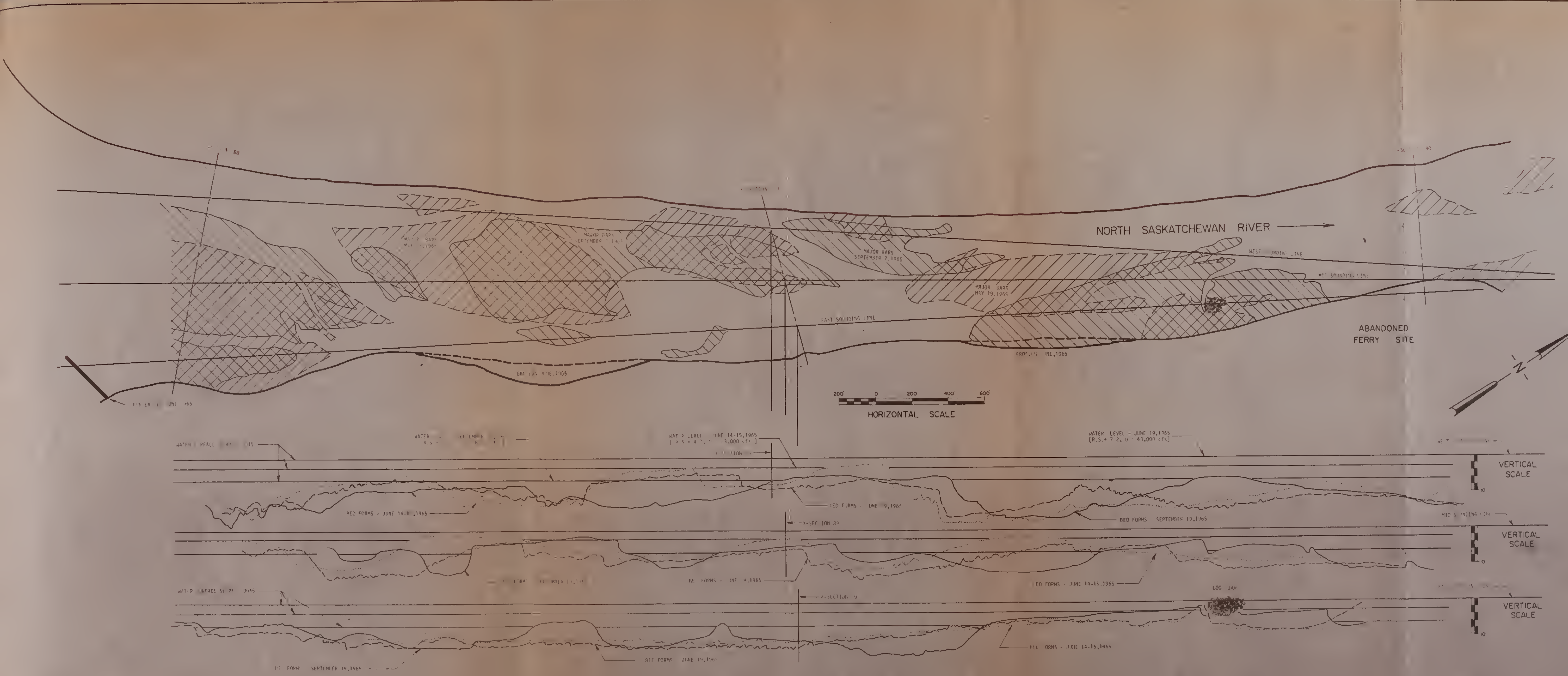


FIGURE 60

UNIT SEDIMENT TRANSPORT VS. MEAN VELOCITY



NORTH SASKATCHEWAN RIVER

SHIFT OF BED FORMS IN STUDY REACH

SUMMER 1965

FIGURE 61

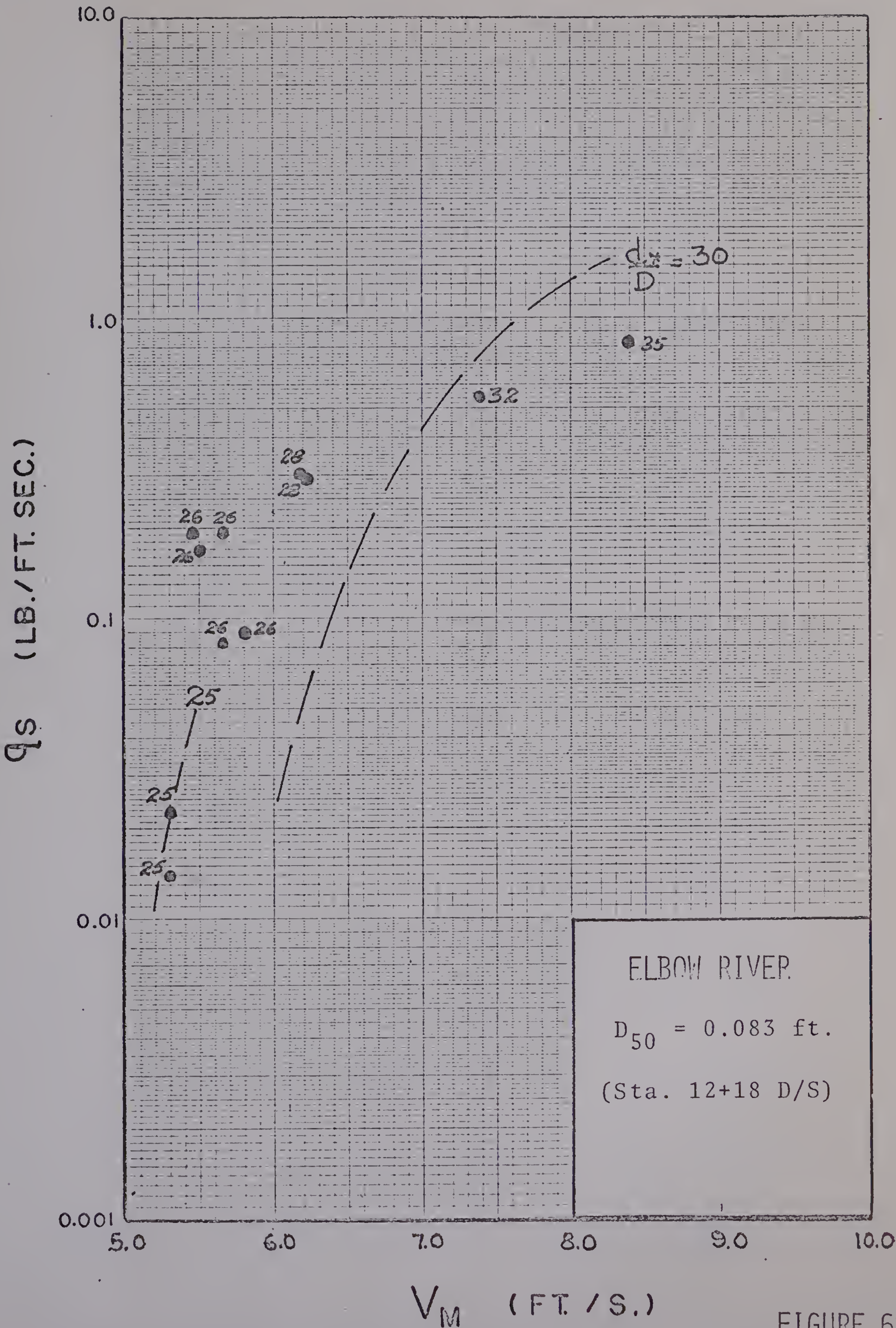


FIGURE 62

UNIT SEDIMENT TRANSPORT VS. MEAN VELOCITY-(ELBOW R.)

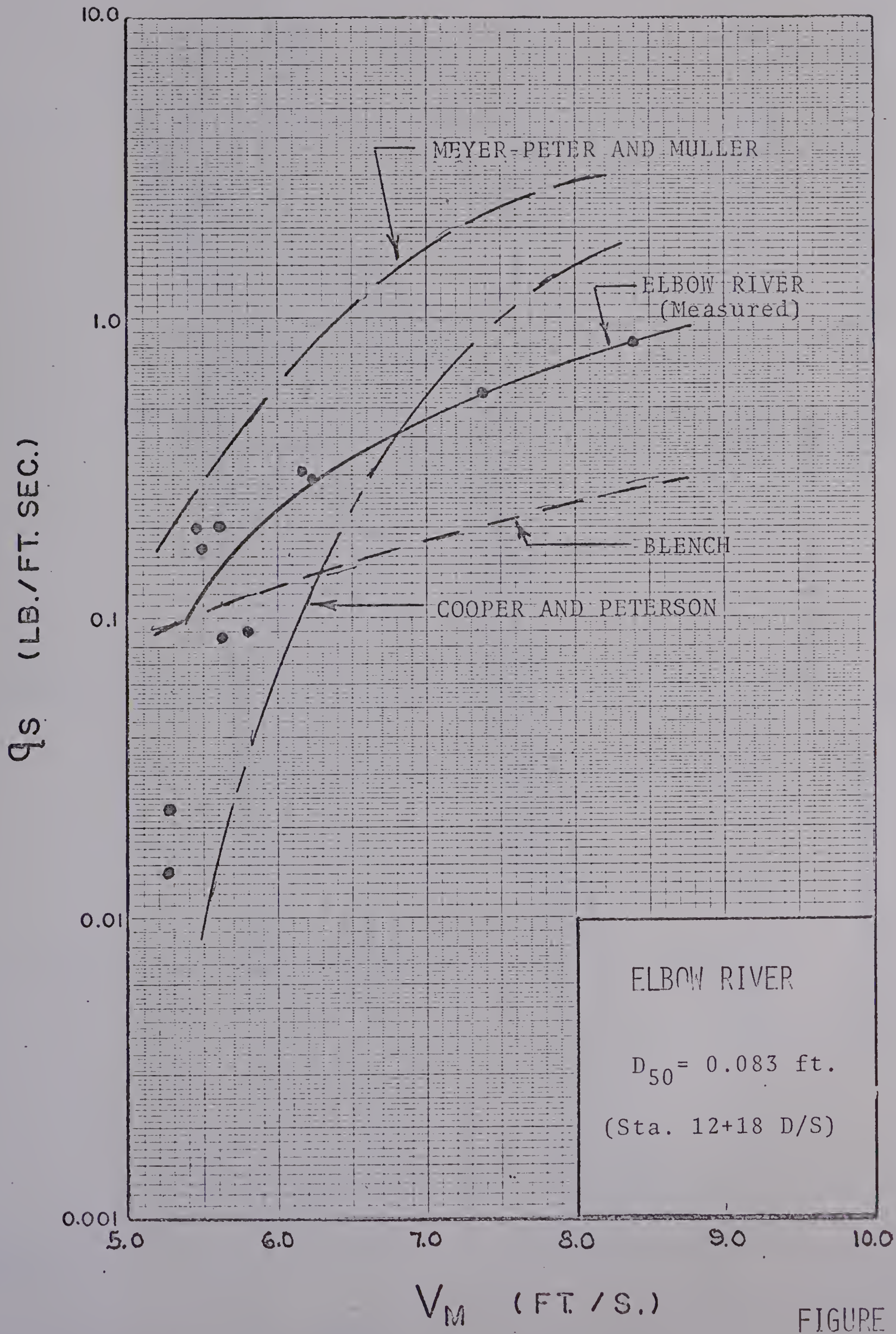
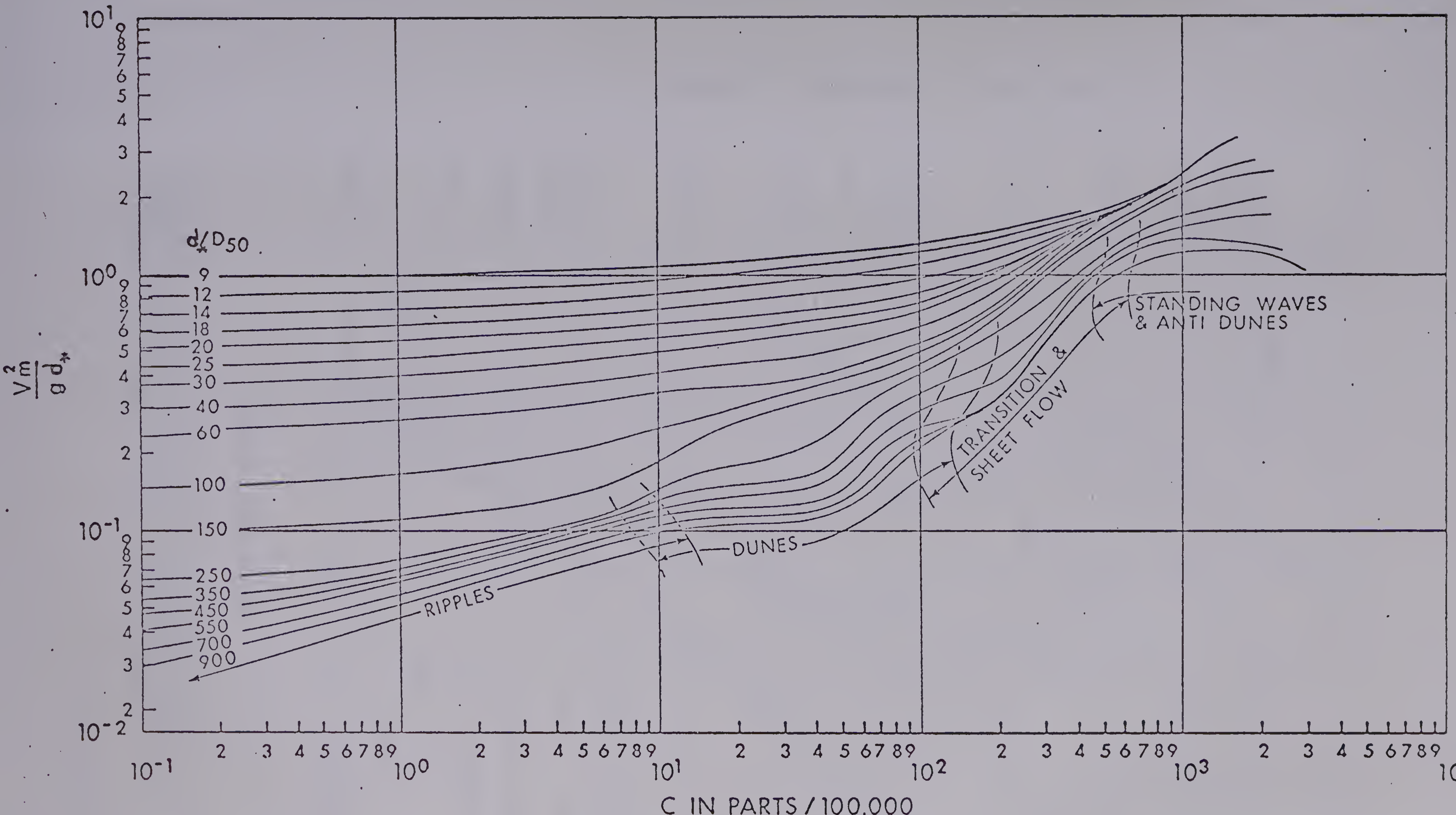


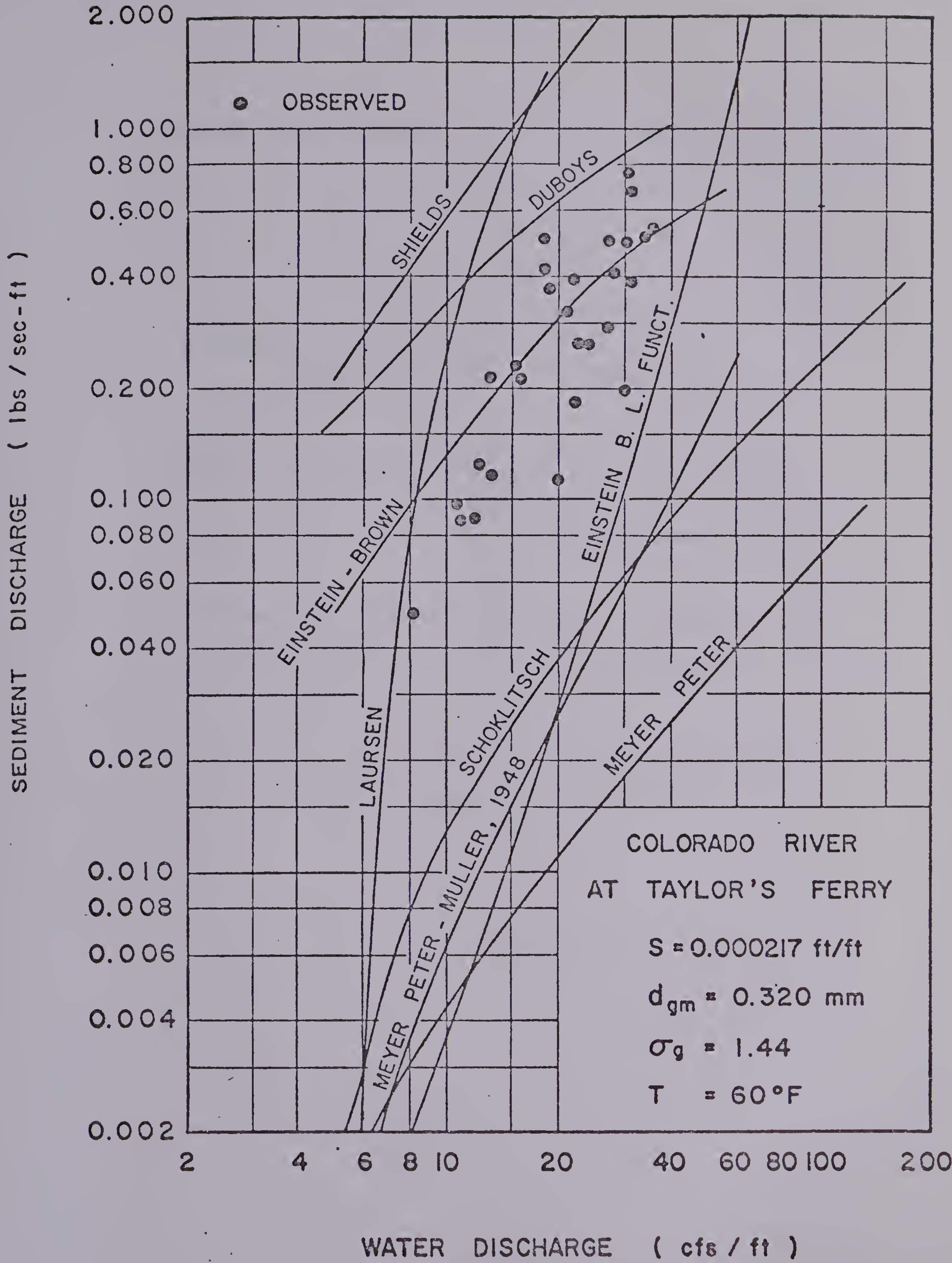
FIGURE 63

COMPARISON OF COMPUTED SEDIMENT TRANSPORT
TO MEASURED TRANSPORT- ELBOW RIVER



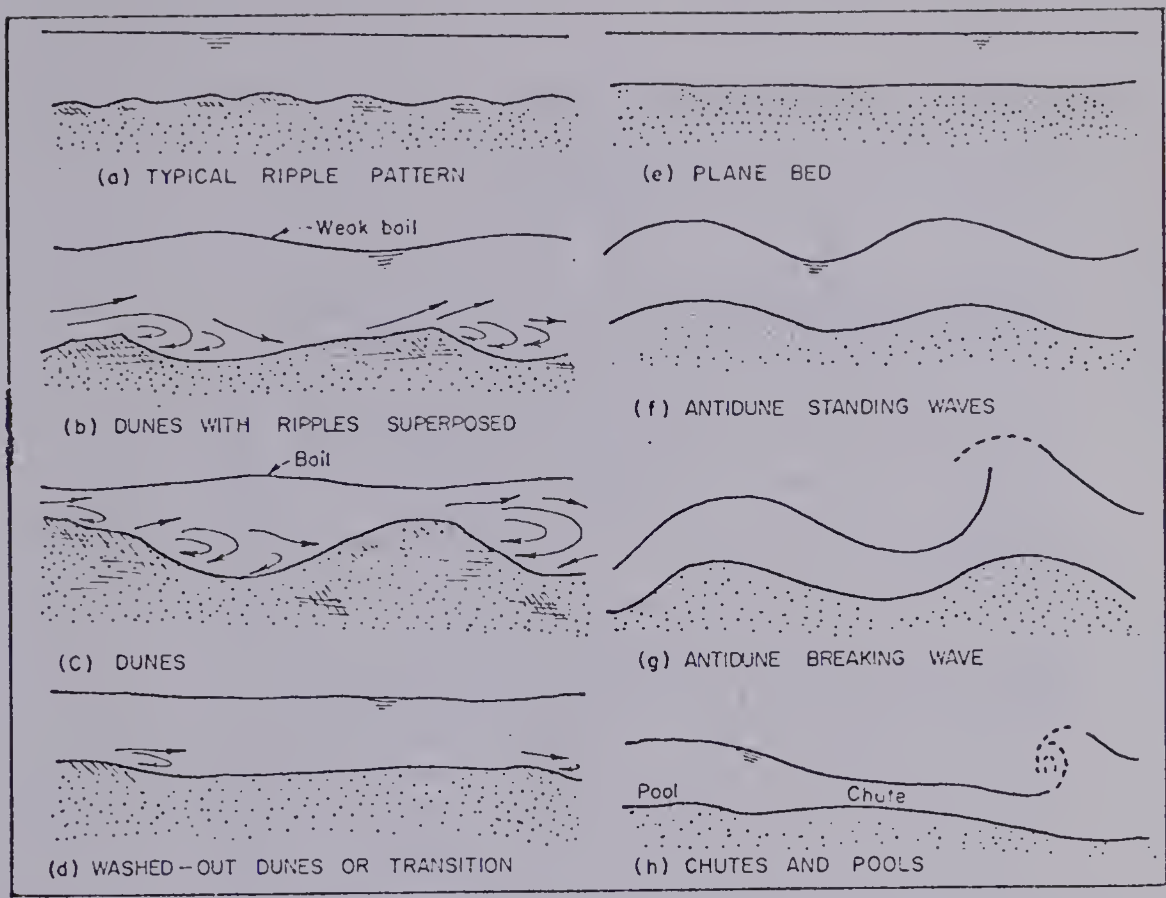
RELATIONSHIP BETWEEN $\frac{V_m^2}{gd_*}$, $\frac{d_*}{D_{50}}$, C (After Cooper and Peterson, 1968)

FIGURE 64



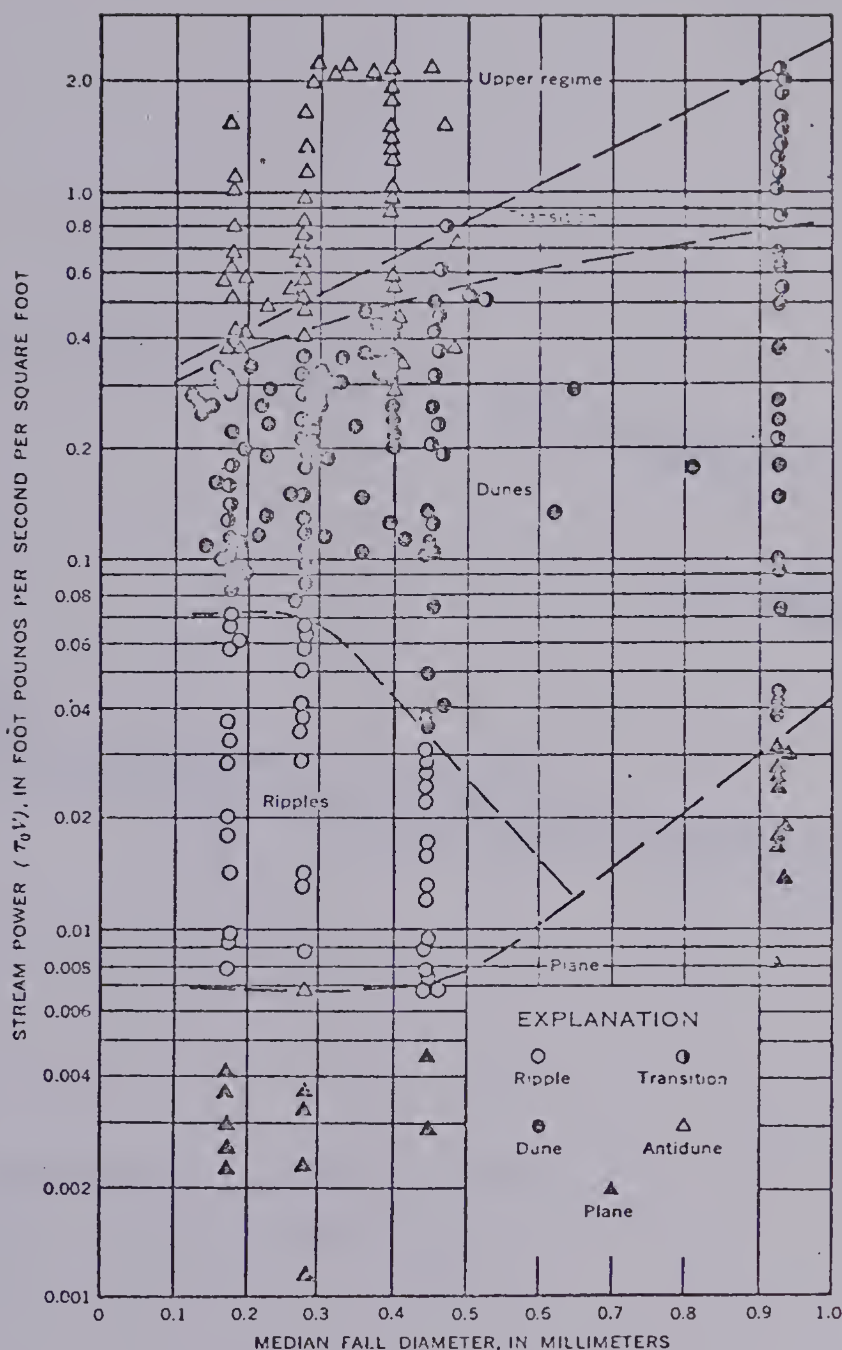
SEDIMENT RATING CURVES FOR COLORADO RIVER AT TAYLOR'S FERRY ACCORDING TO SEVERAL TRANSPORT FORMULAS, COMPARED WITH MEASUREMENTS (VANONI, BROOKS AND KENNEDY).

FIGURE 65



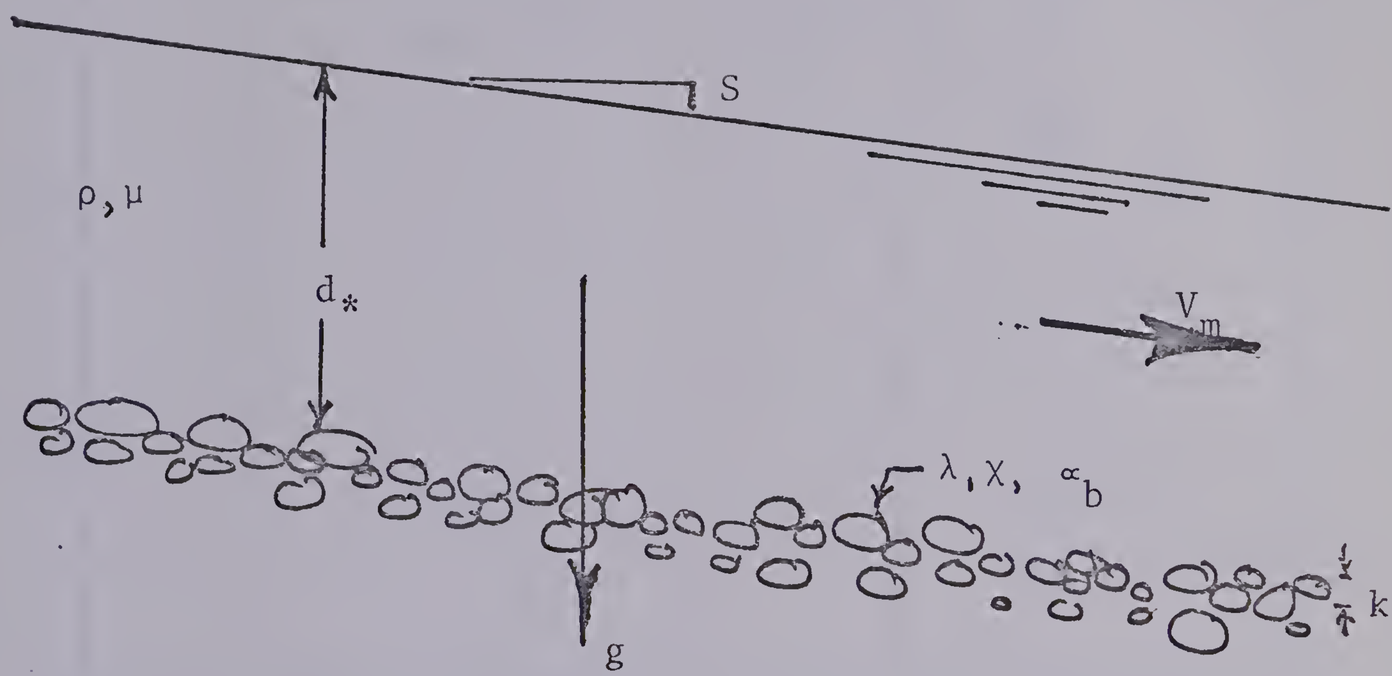
IDEALIZED BED FORMS IN AN ALLUVIAL CHANNEL
(After Simons and Richardson, 1961)

FIGURE 66



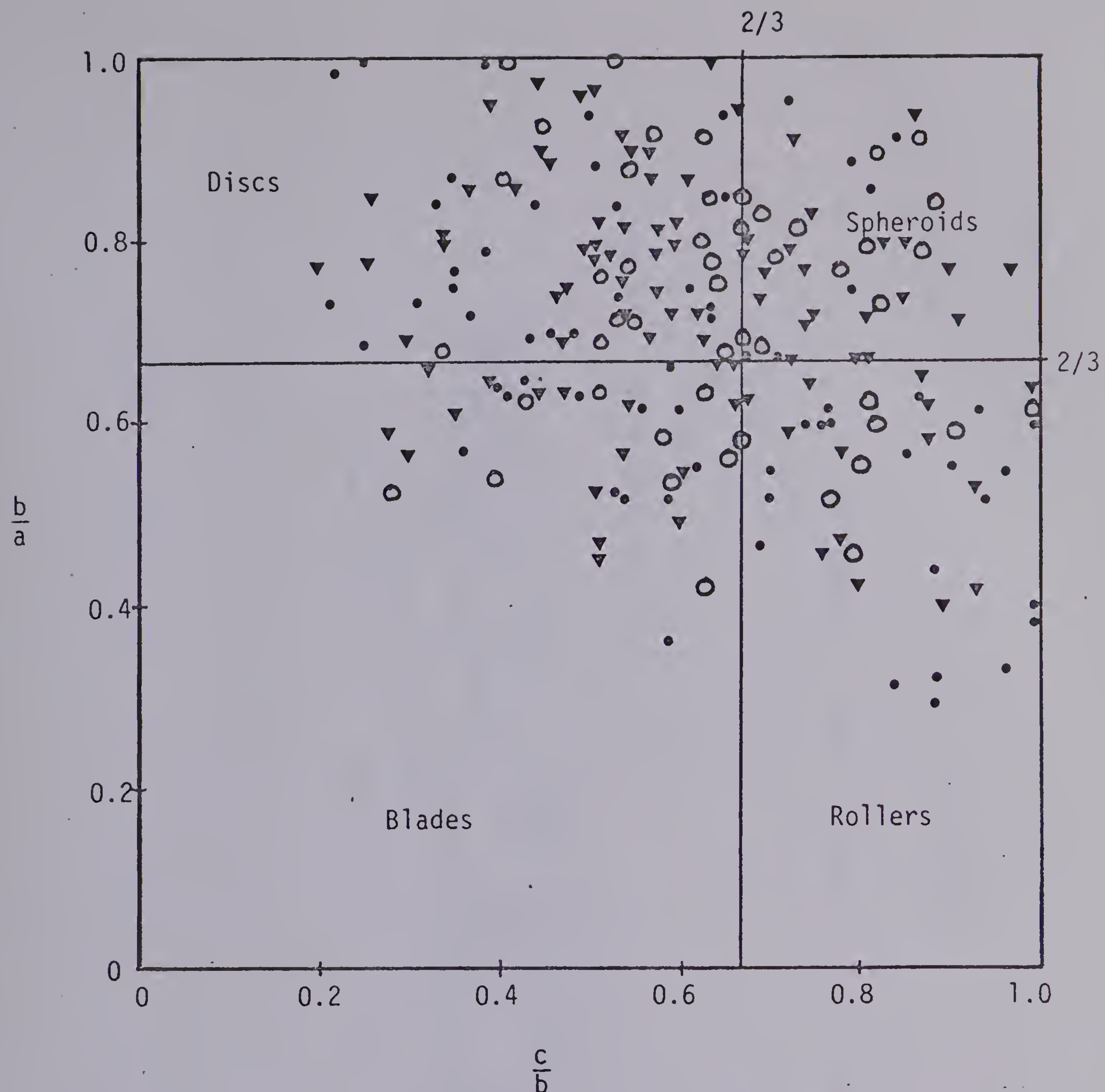
RELATION OF BED FORM TO STREAM POWER
AND MEDIAN FALL DIAMETER
(After Simons and Richardson,1966)

FIGURE 67



INDEPENDENT VARIABLES FOR AN
IMMOBILE COARSE CHANNEL BED

FIGURE 68

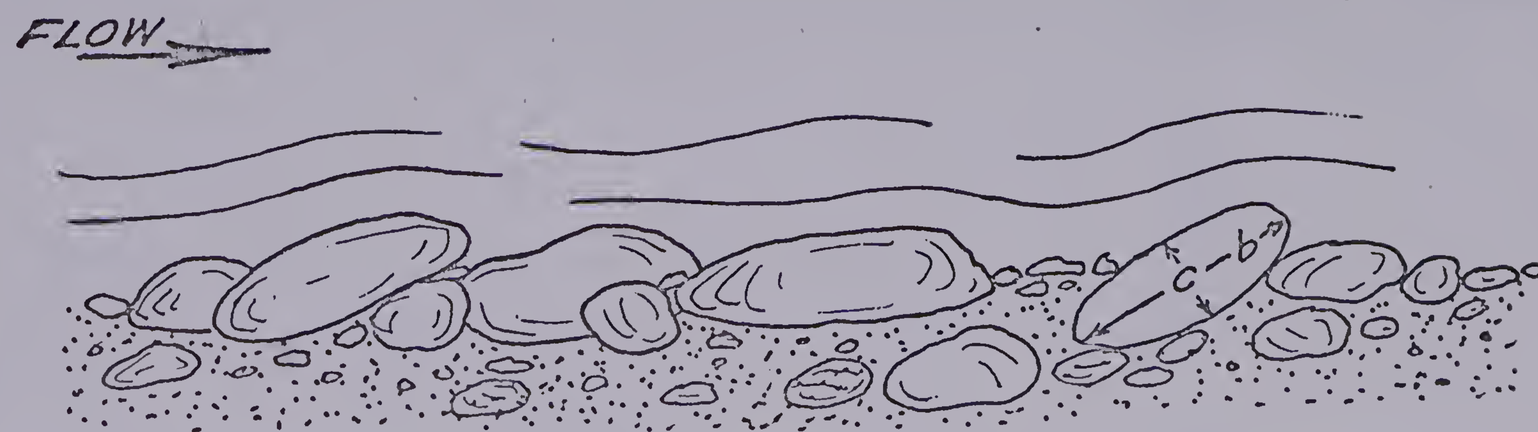


SHAPE RATIOS OF BED MATERIAL

a - long axis
b - intermediate axis
c - short axis

Sample #36, b > 3" •
Sample #58, b > 3" ▼ ELBOW RIVER
Cross-Sec. A2- Qureshi RED DEER R.

FIGURE 69



b = intermediate axis
c = minor axis

ARRANGEMENT OF COARSE BED PARTICLES

FIGURE 70

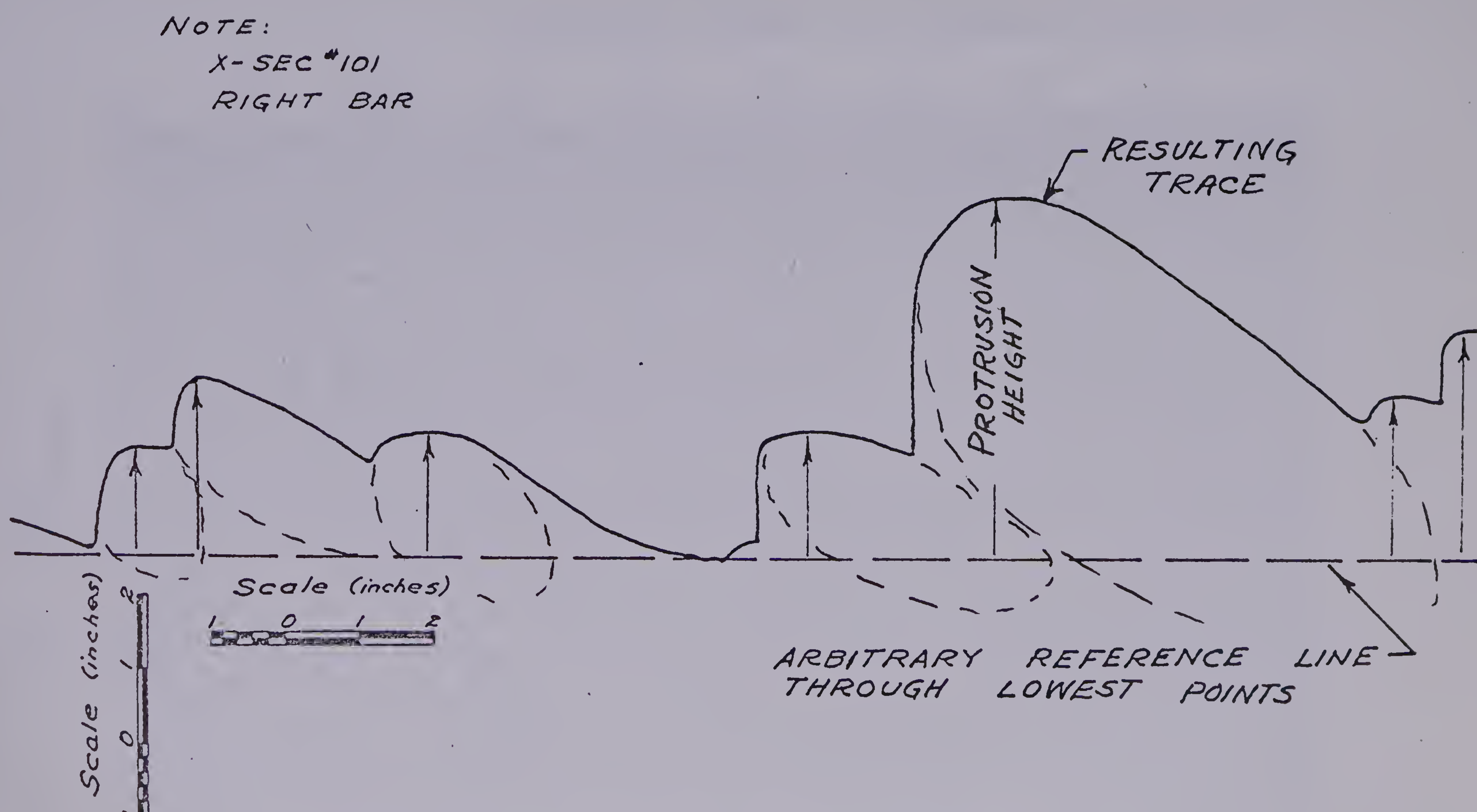
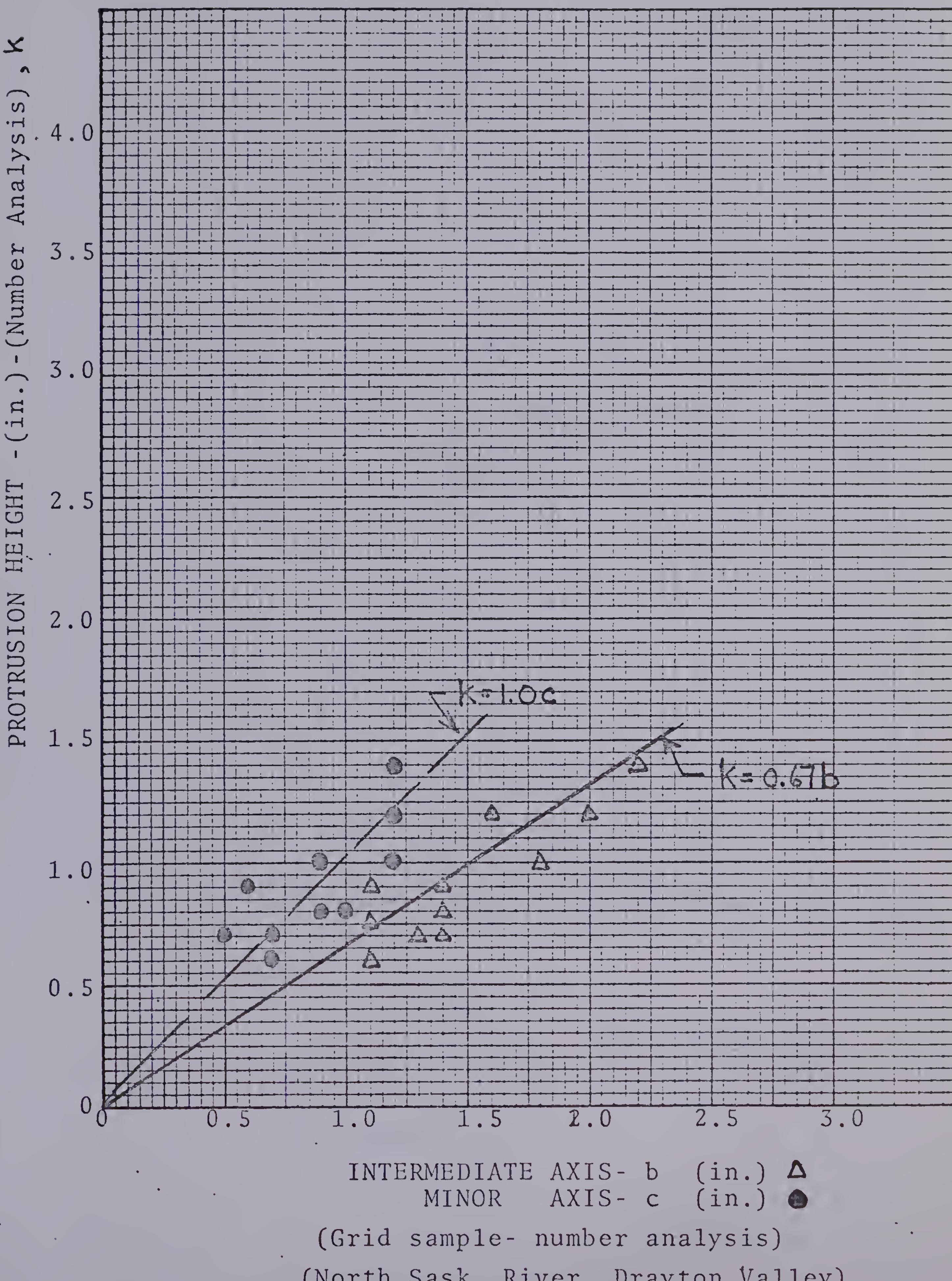


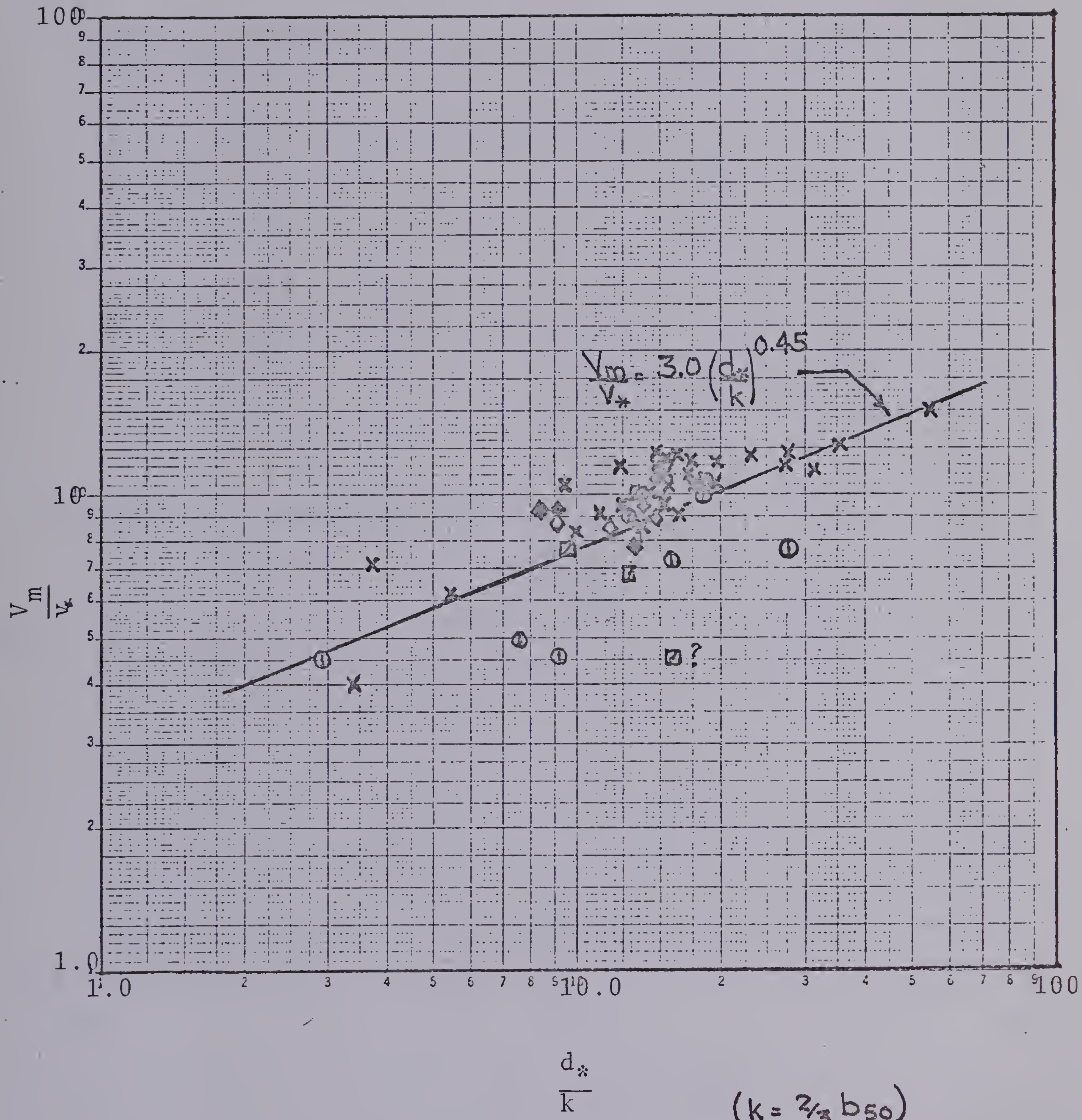
CHART FROM BED ROUGHNESS METER

FIGURE 71



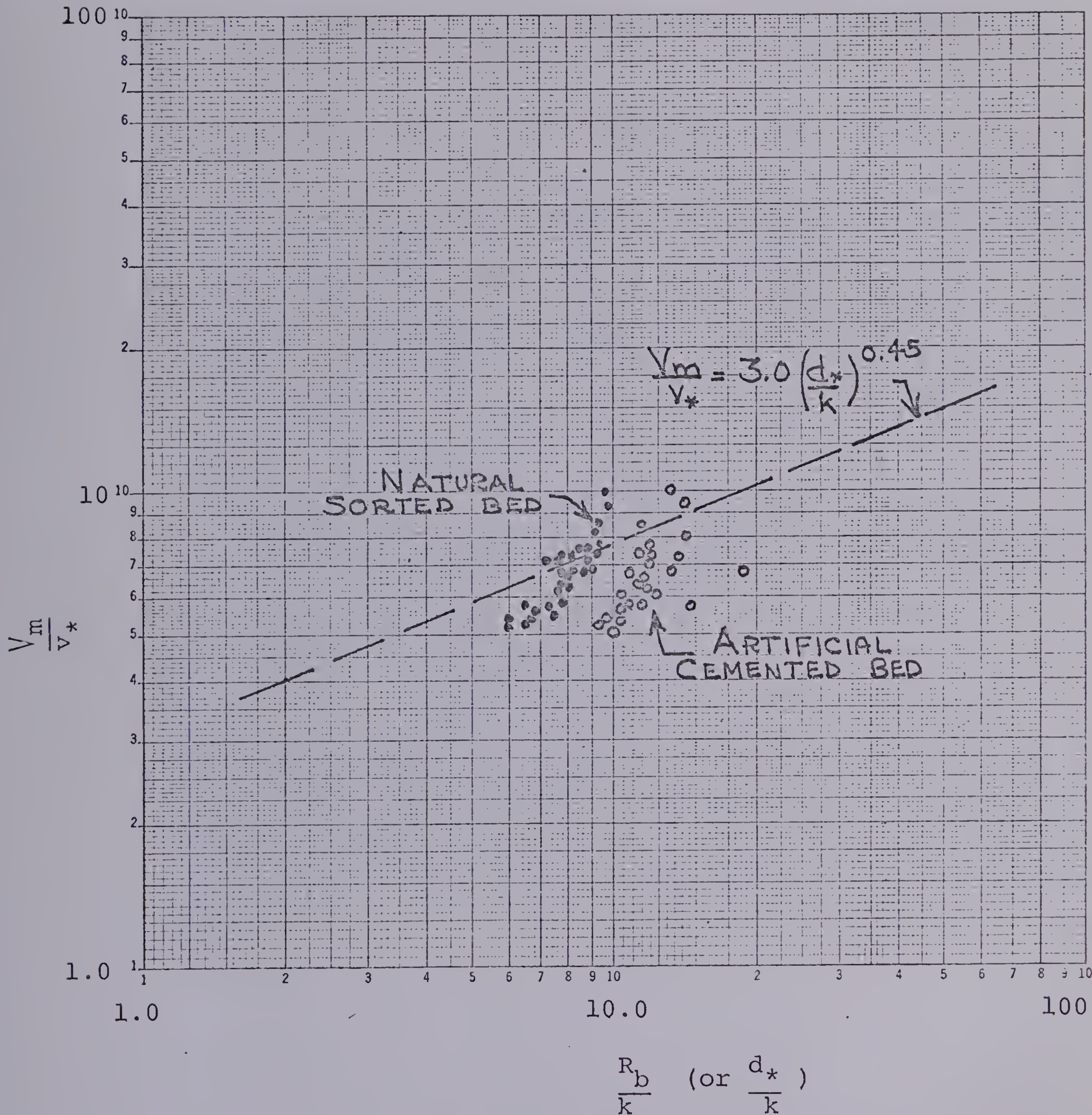
PROTRUSION HEIGHT VERSUS INTERMEDIATE AND MINOR AXIS

FIGURE 72



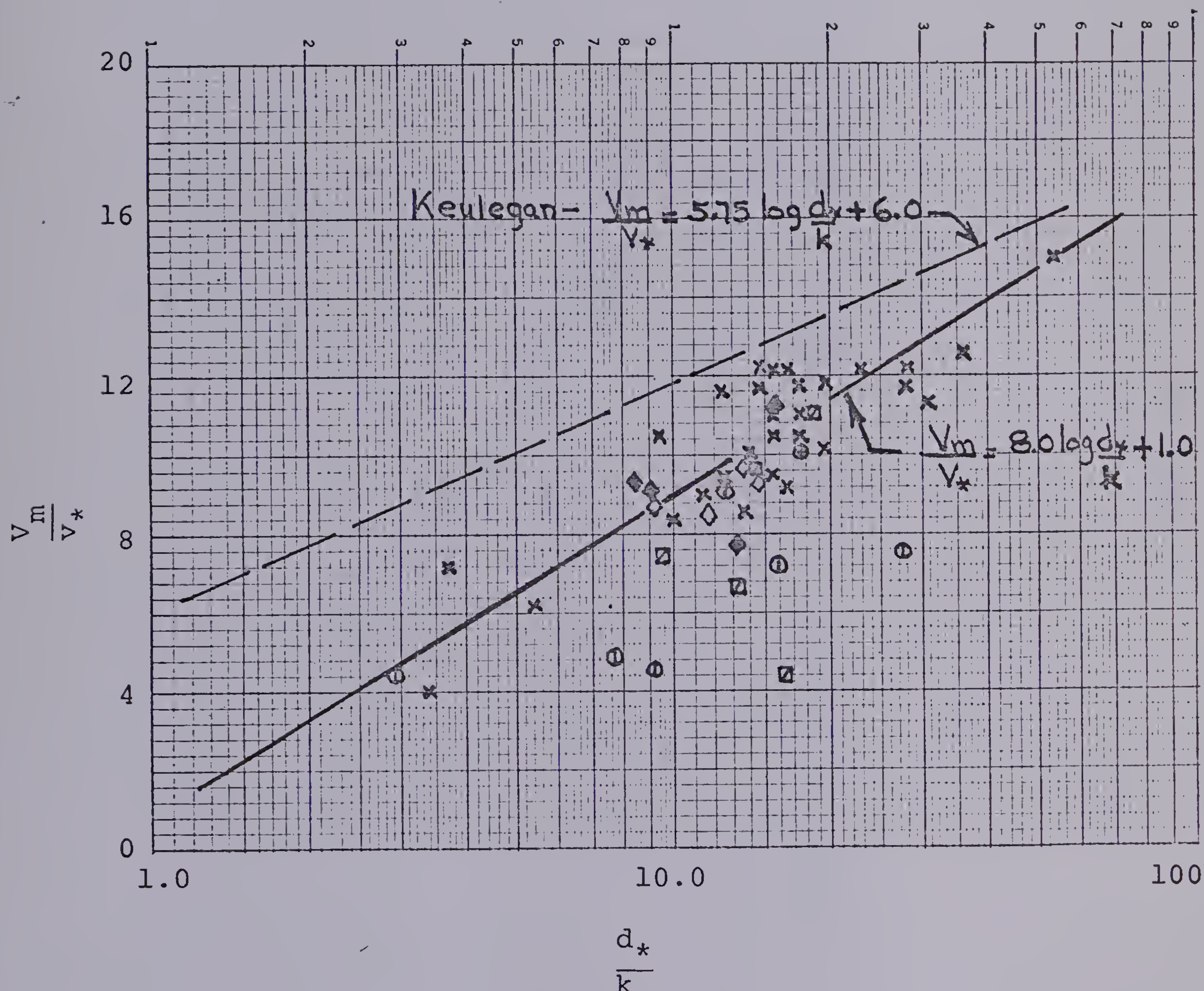
$\frac{V_m}{v_*}$ VERSUS $\frac{d_*}{k}$

FIGURE 73



$\frac{V_m}{v_*}$ VERSUS $\frac{R_b}{k}$ FOR FLUME TESTS

FIGURE 74



$\frac{V_m}{v_*}$ VERSUS $\frac{d_*}{k}$ (Log-arithmetic Plot)

FIGURE 75

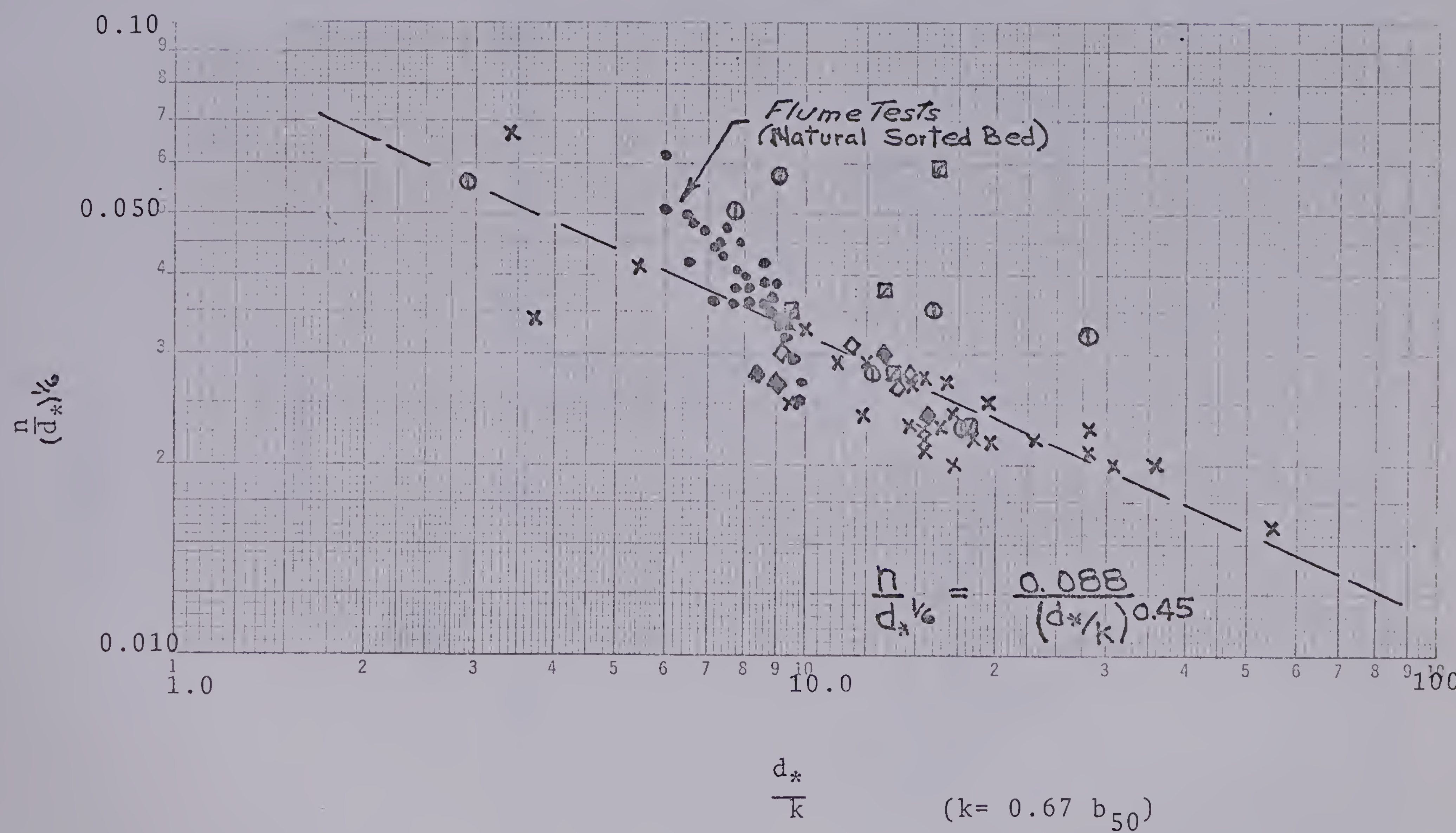
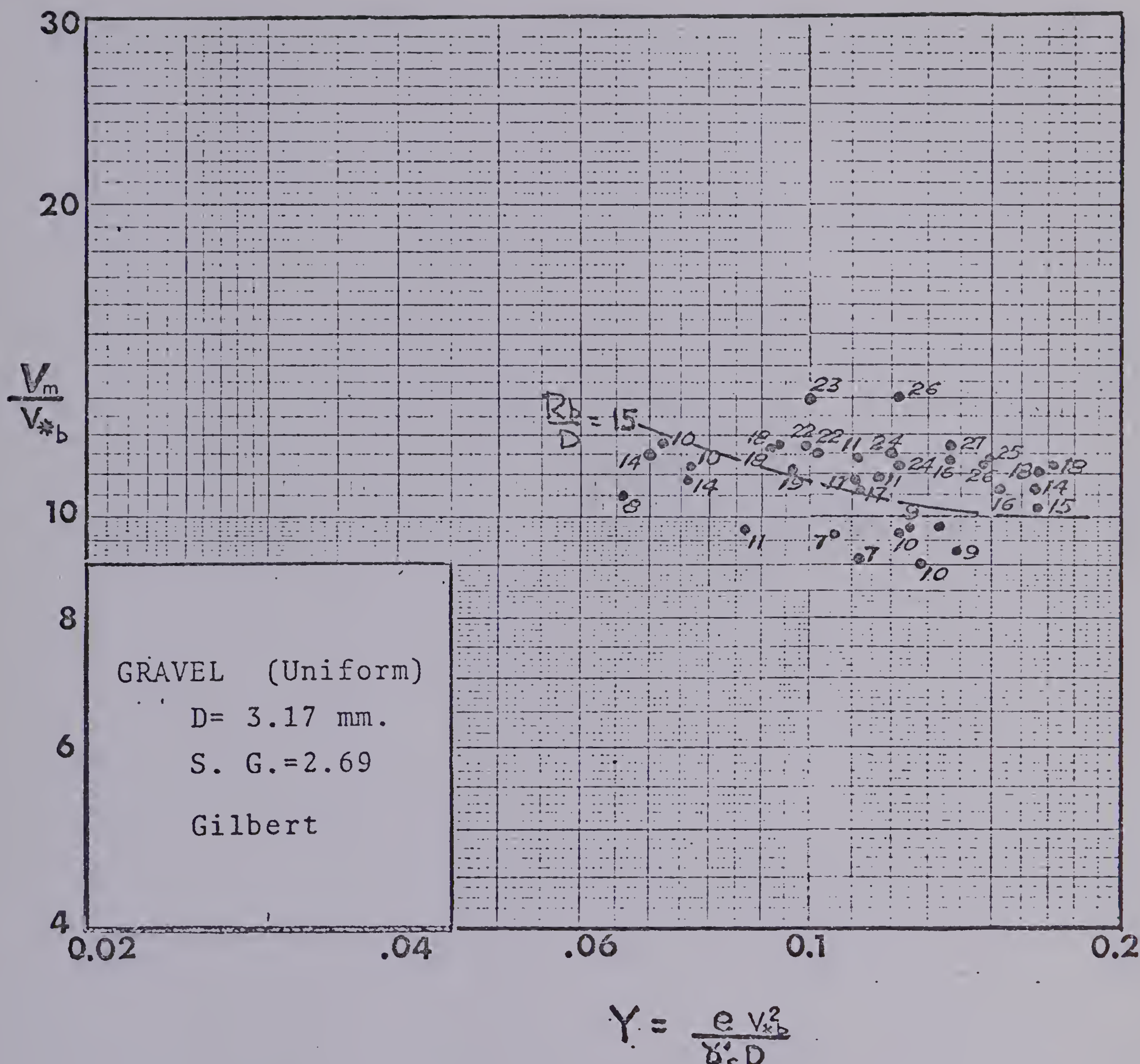
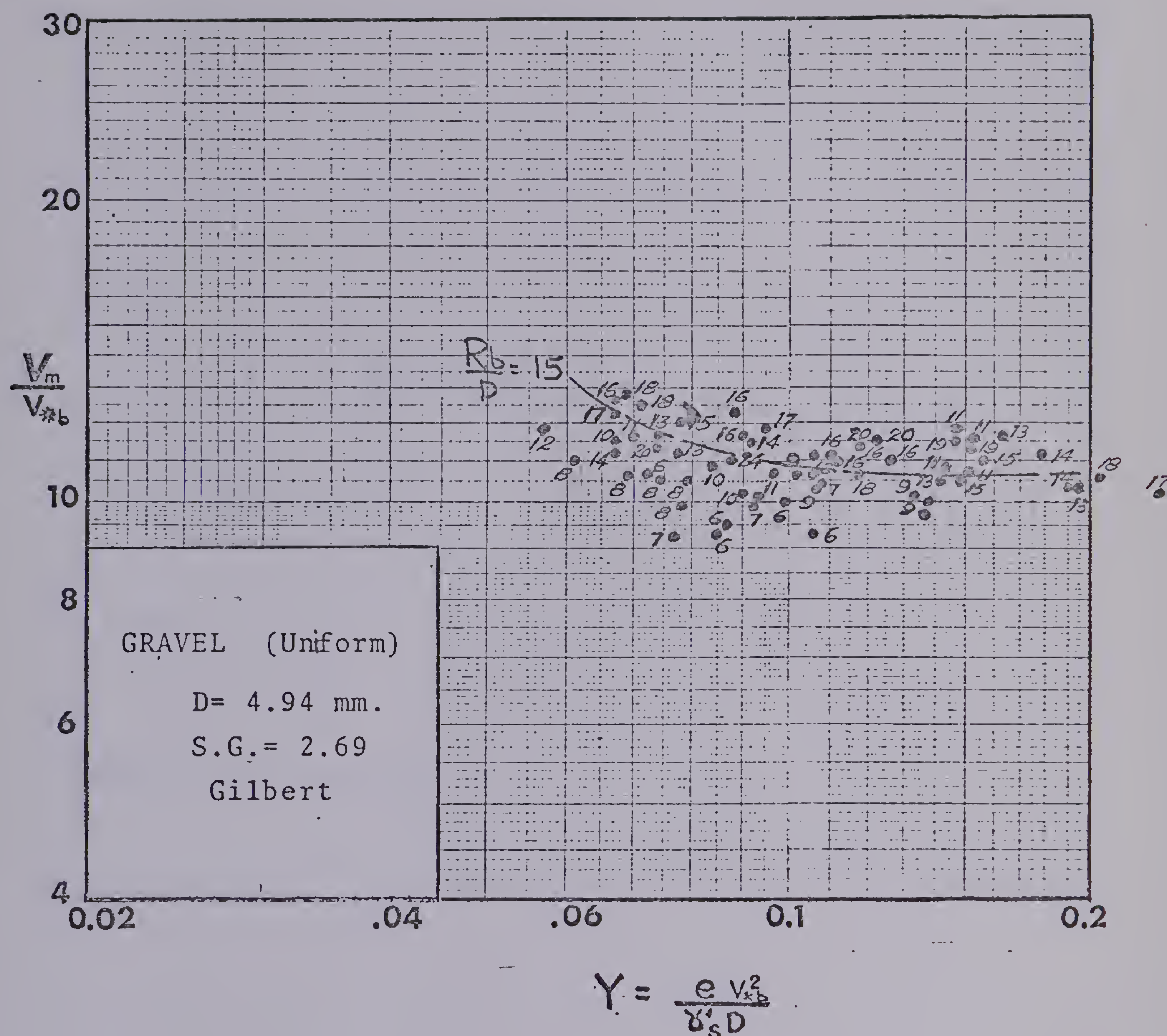

0.10
0.050
0.010
1.0
10.0
100
Flume Tests
(Natural Sorted Bed)
$\frac{n}{d_*^{1/6}} = \frac{0.088}{(d_*/k)^{0.45}}$
$\frac{n}{(d_*)^{1/6}}$
$\frac{d_*}{k}$
(k= 0.67 b_{50})

FIGURE 76



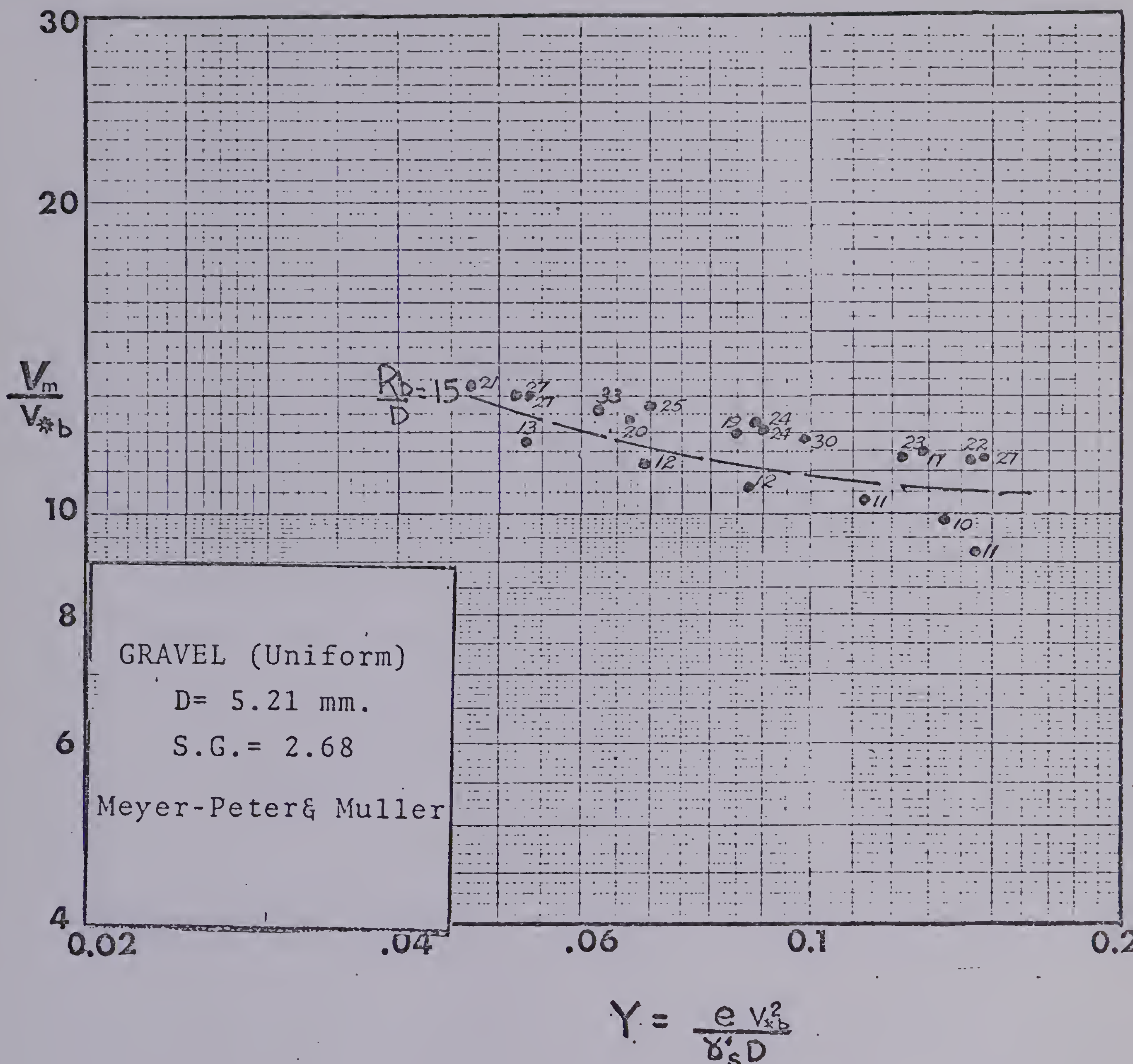
$\frac{V_m}{V_{*b}}$ VERSUS. Y

FIGURE 77



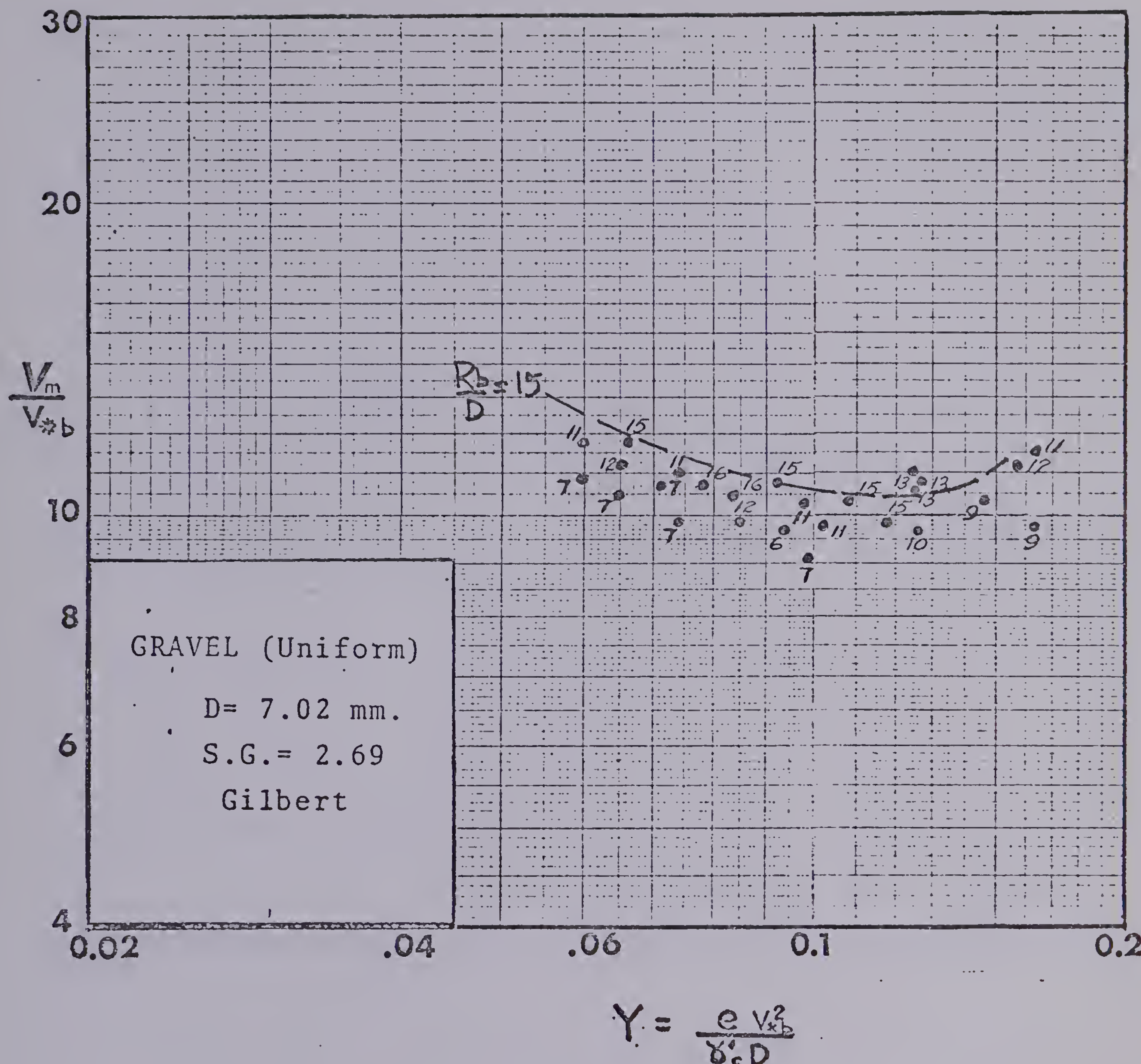
$\frac{V_m}{V_{*b}}$ VERSUS Y

FIGURE 78



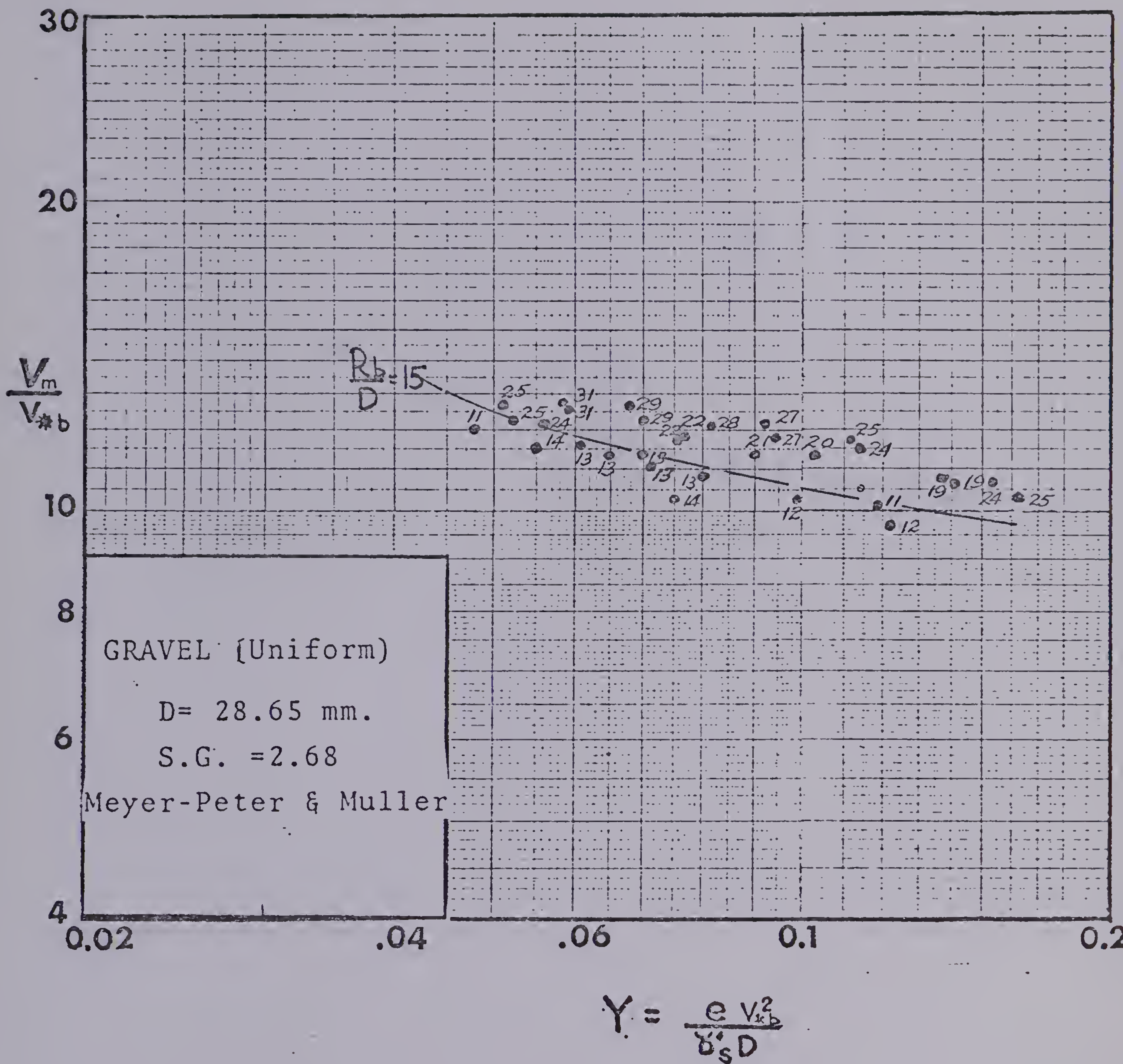
$\frac{V_m}{V_{*b}}$ VERSUS Y

FIGURE 79



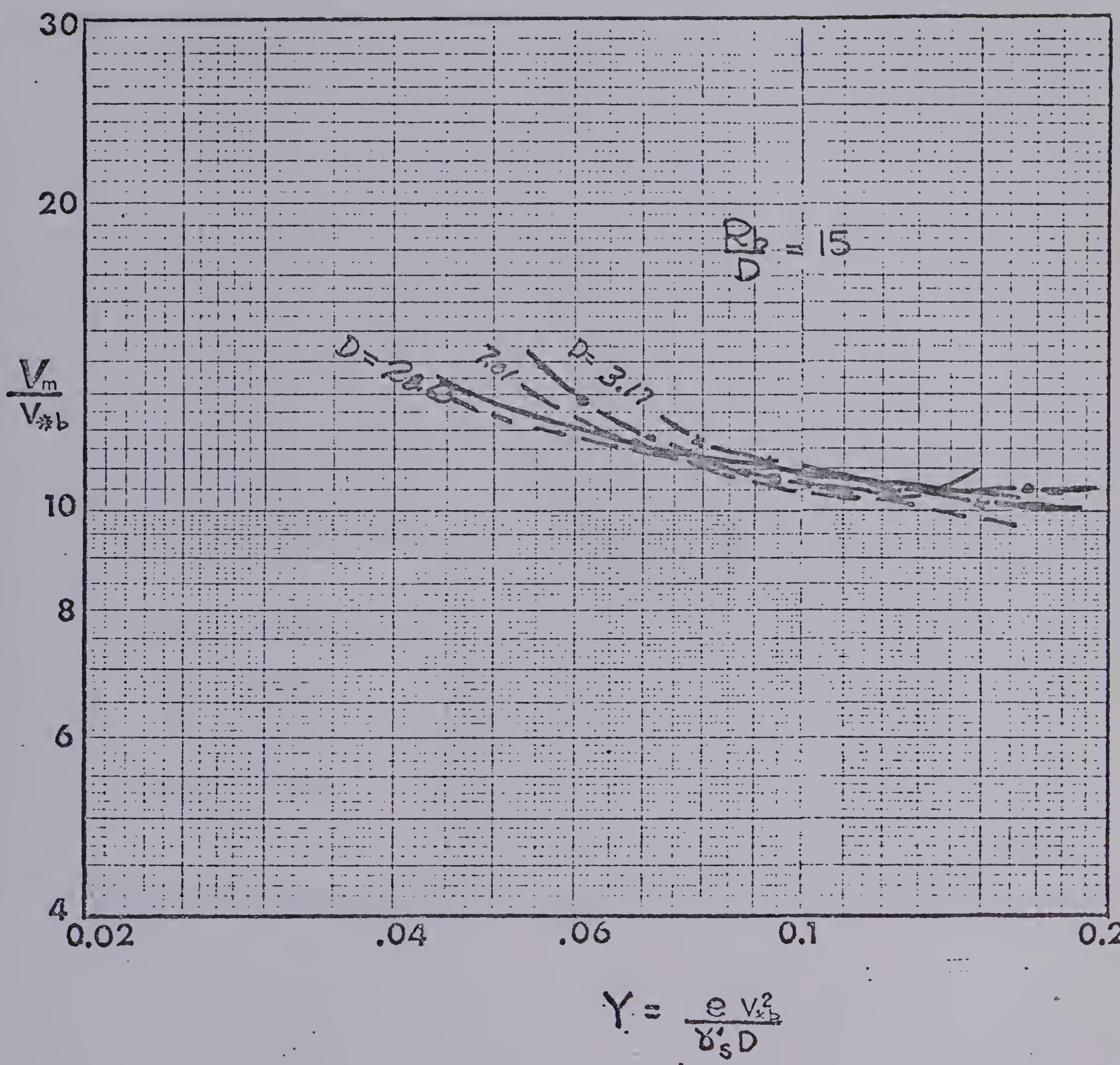
$\frac{V_m}{v_{*b}}$ VERSUS Y

FIGURE 80



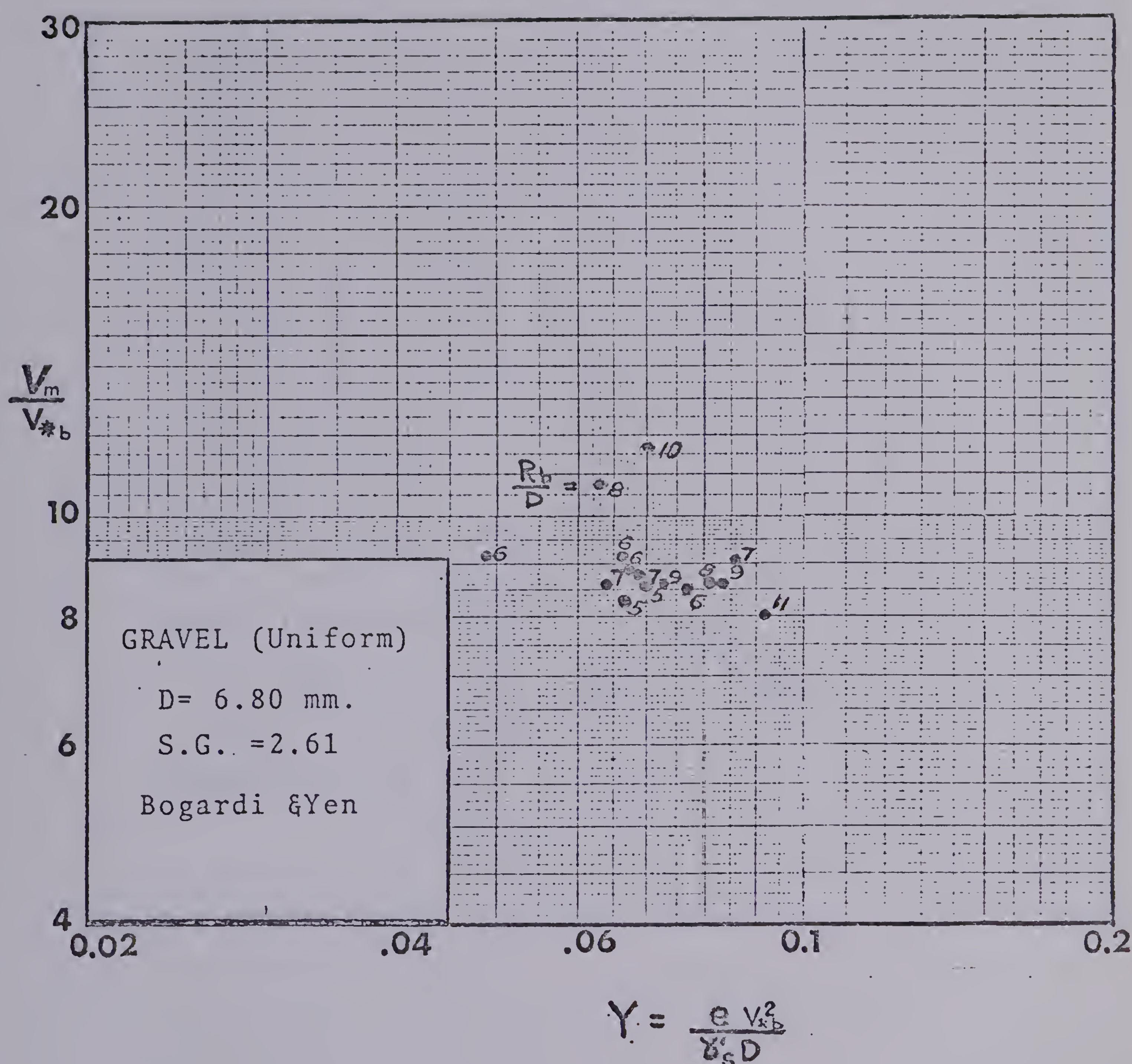
$\frac{V_m}{V_{*b}}$ VERSUS Y

FIGURE 81



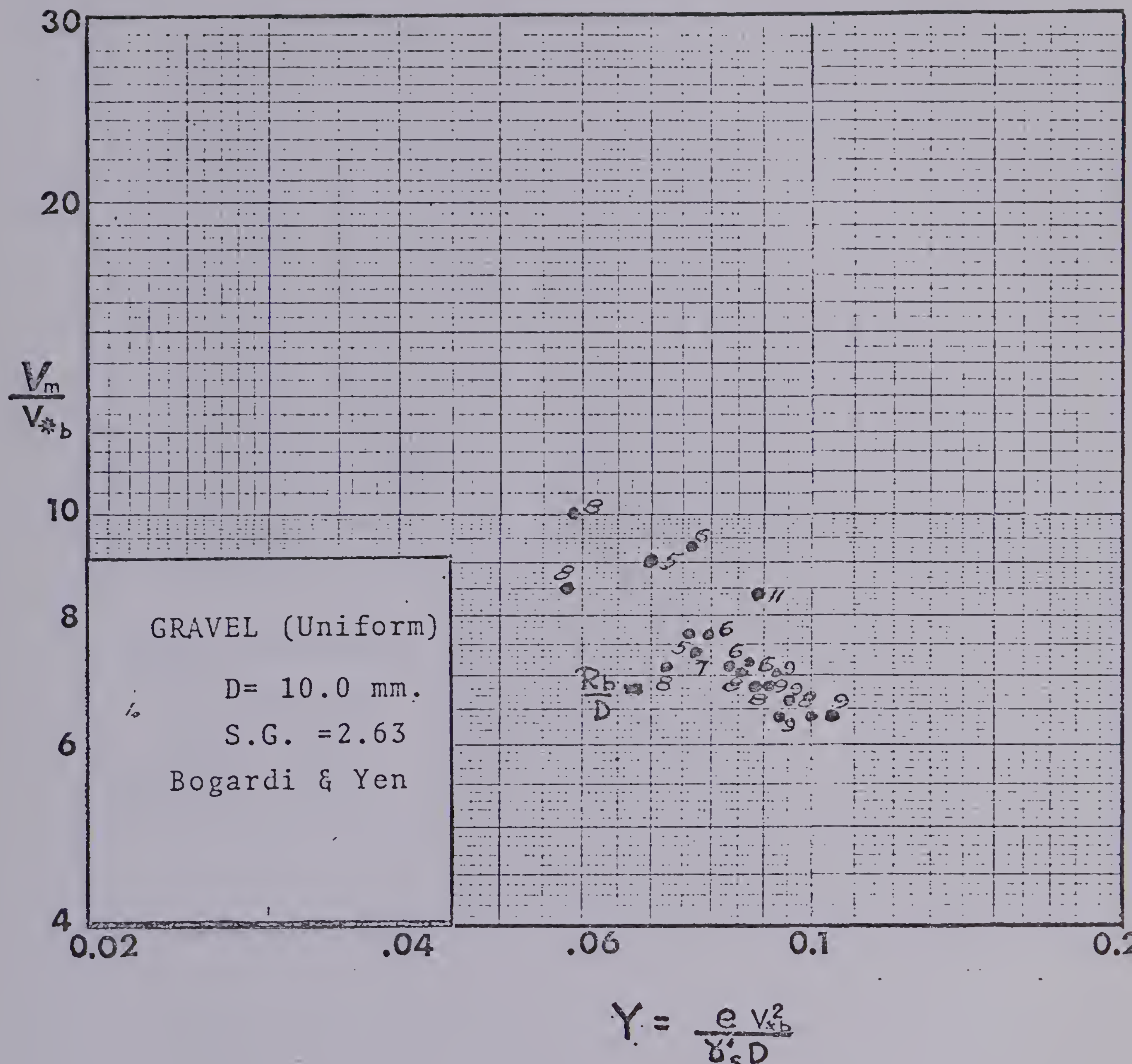
$\frac{V_m}{v_{*_b}}$ VERSUS Y FOR $\frac{R_b}{D} = 15$

FIGURE 82



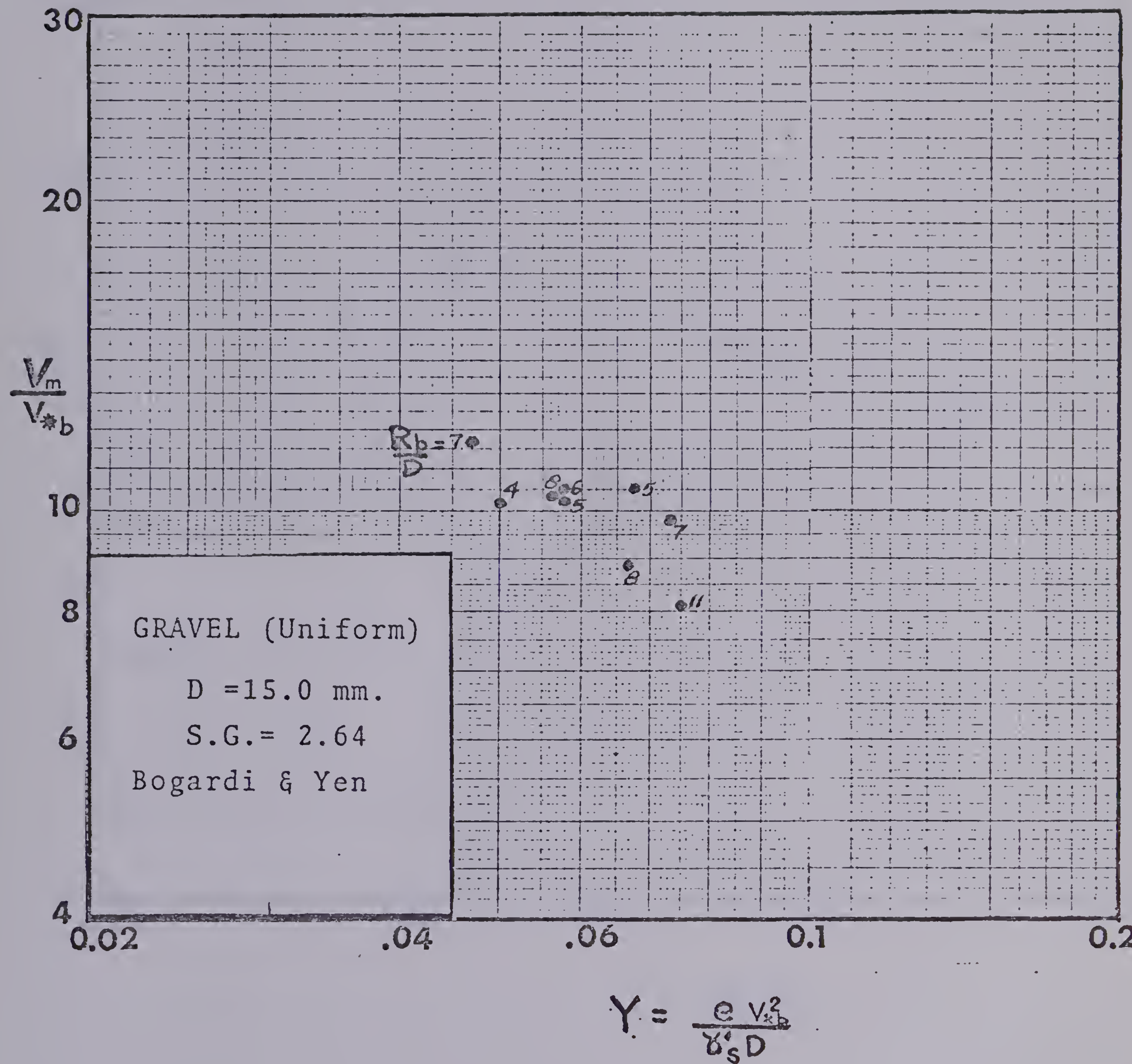
$\frac{V_m}{V_{*_b}}$ VERSUS Y

FIGURE 83



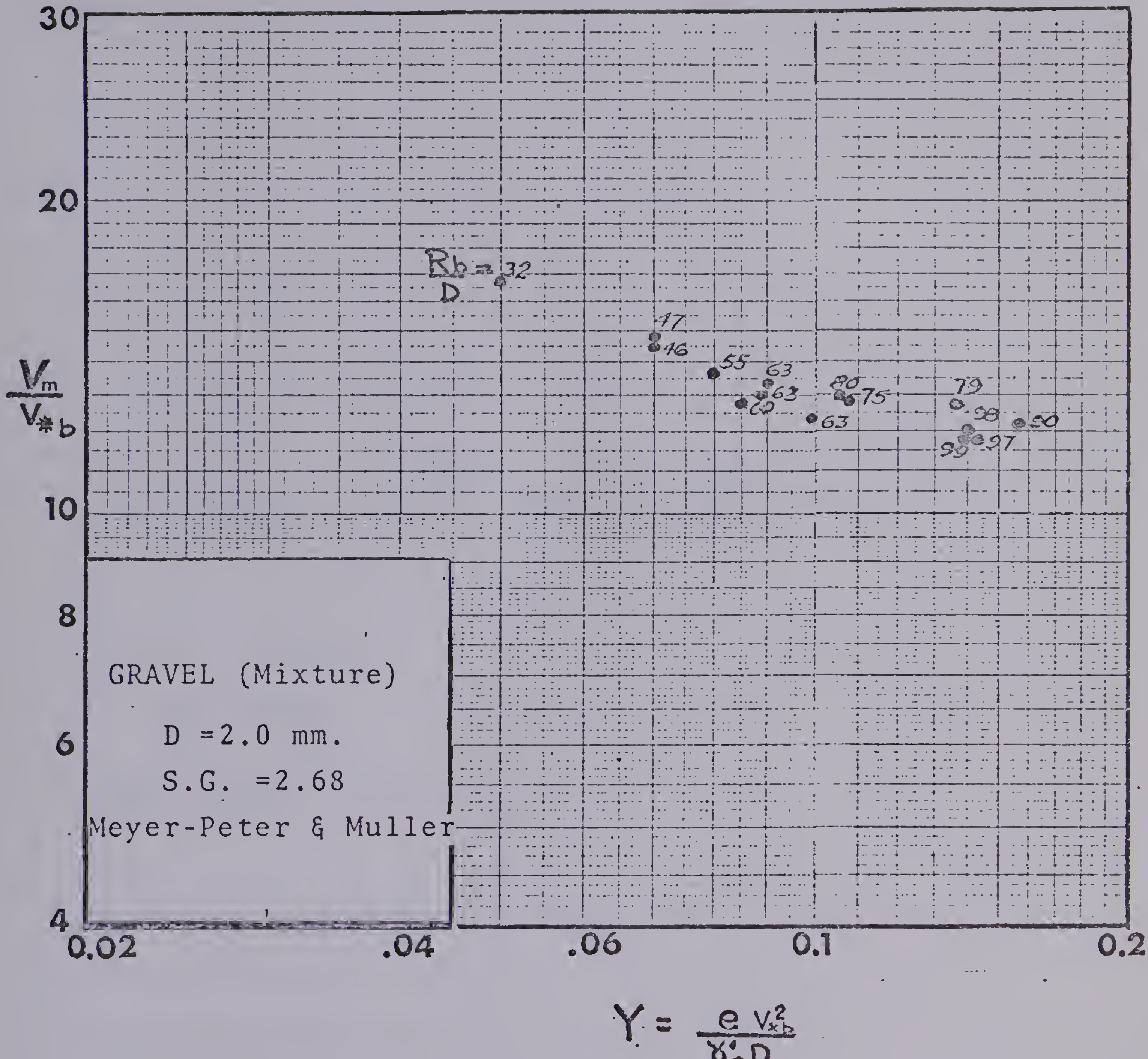
$\frac{V_m}{V_{*b}}$ VERSUS Y

FIGURE 84



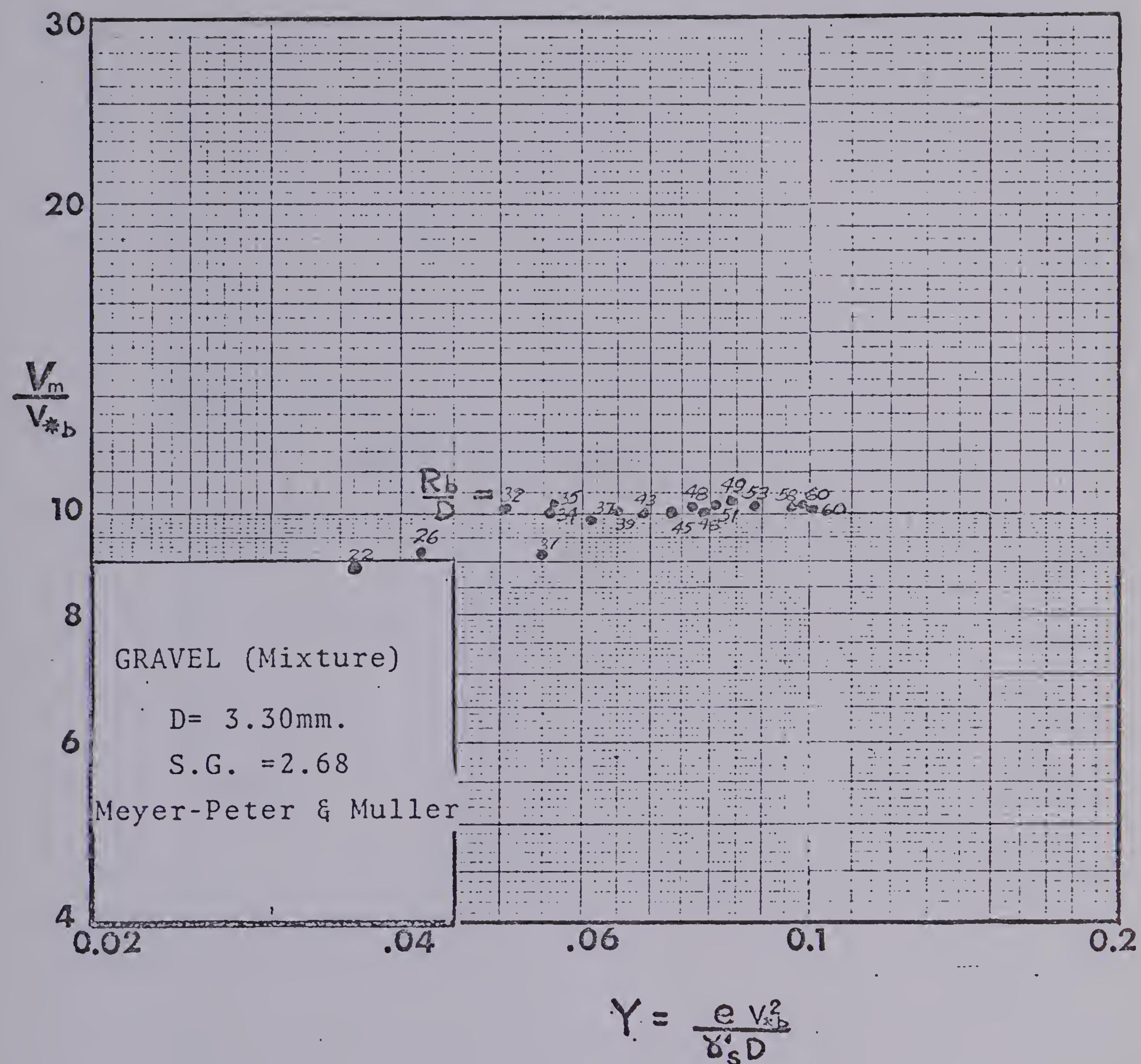
$\frac{V_m}{V_{*b}}$ VERSUS Y

FIGURE 85



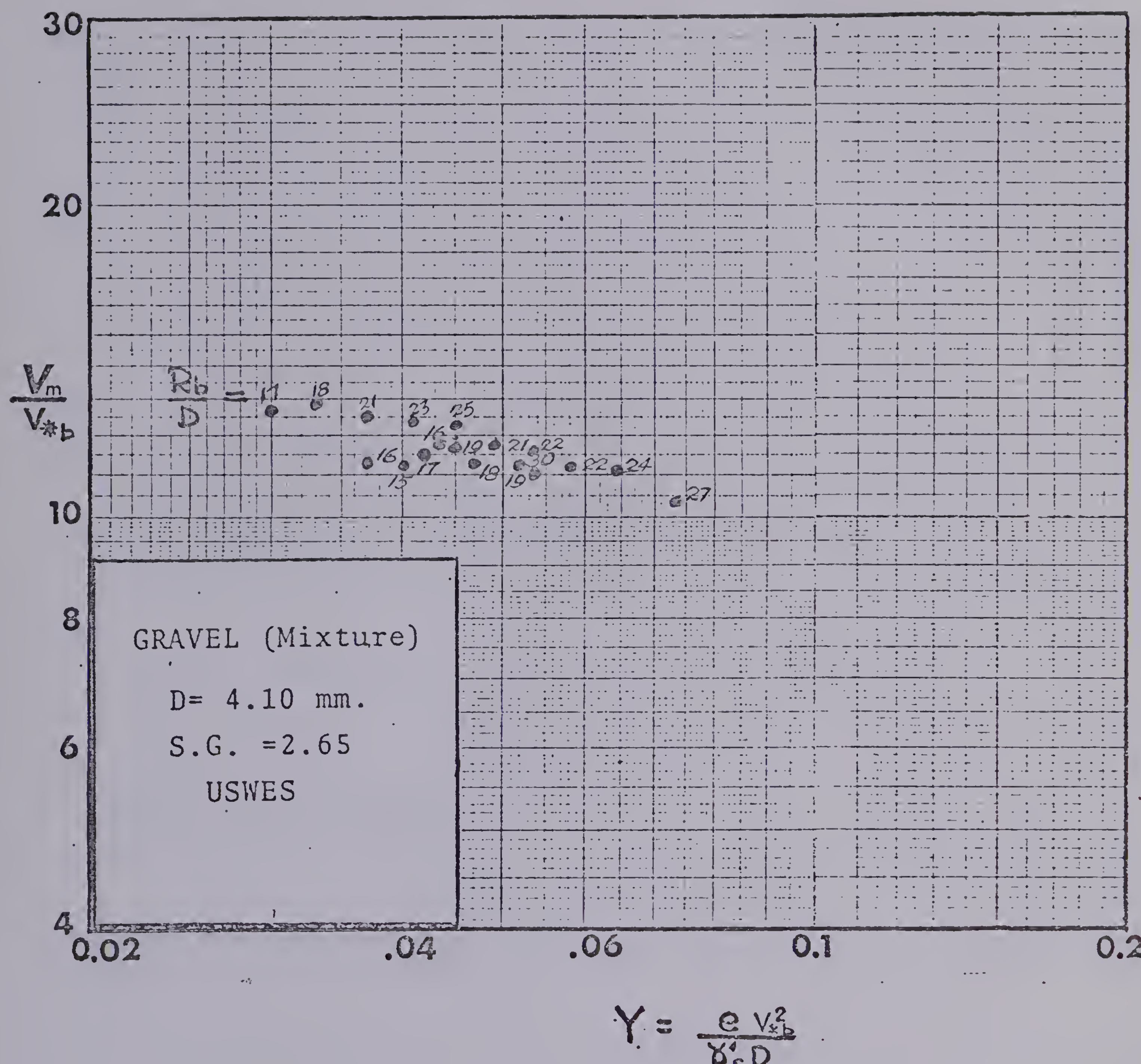
$\frac{V_m}{V_{*b}}$ VERSUS Y

FIGURE 86



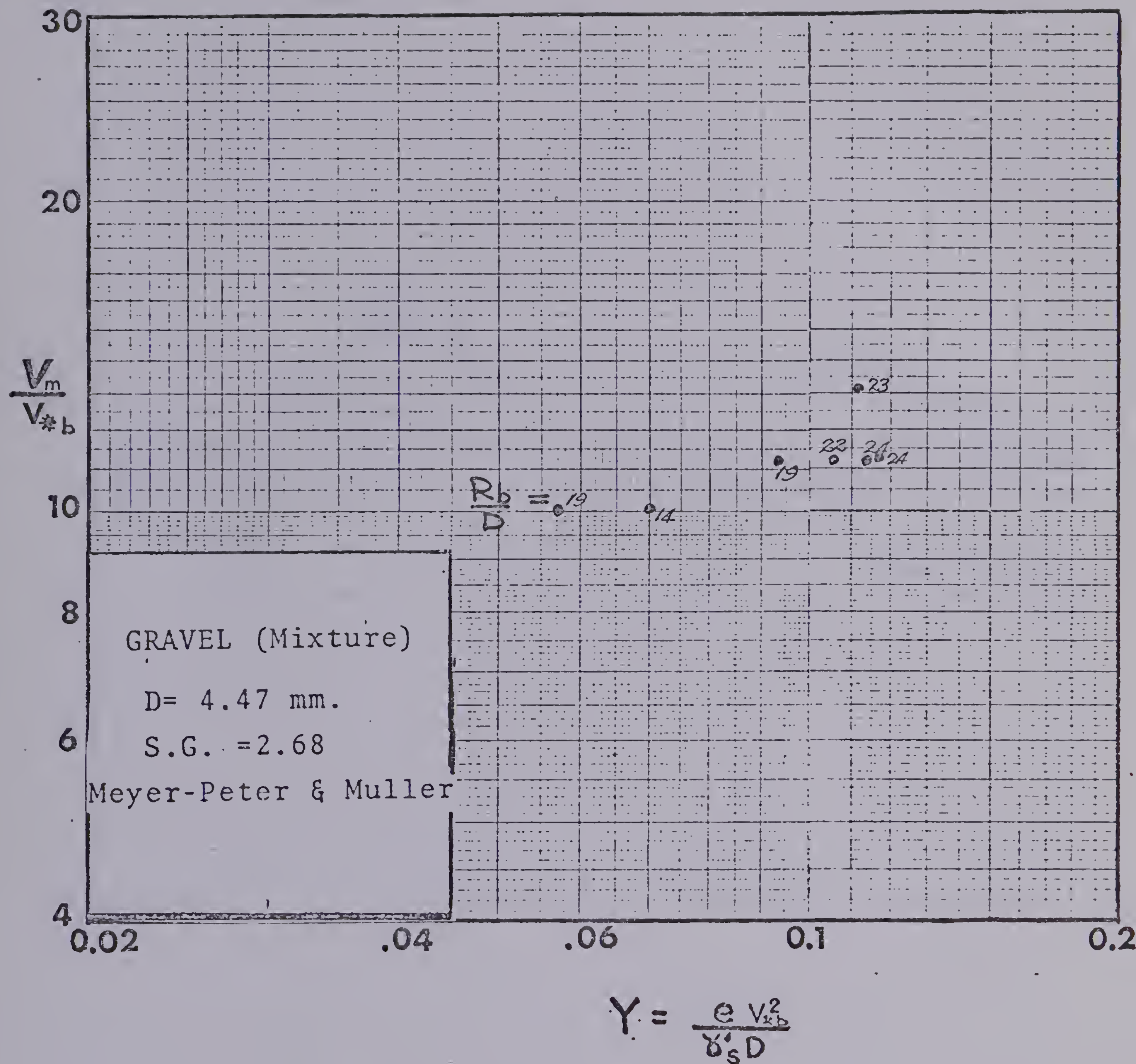
$\frac{V_m}{V_{*b}}$ VERSUS Y

FIGURE 87



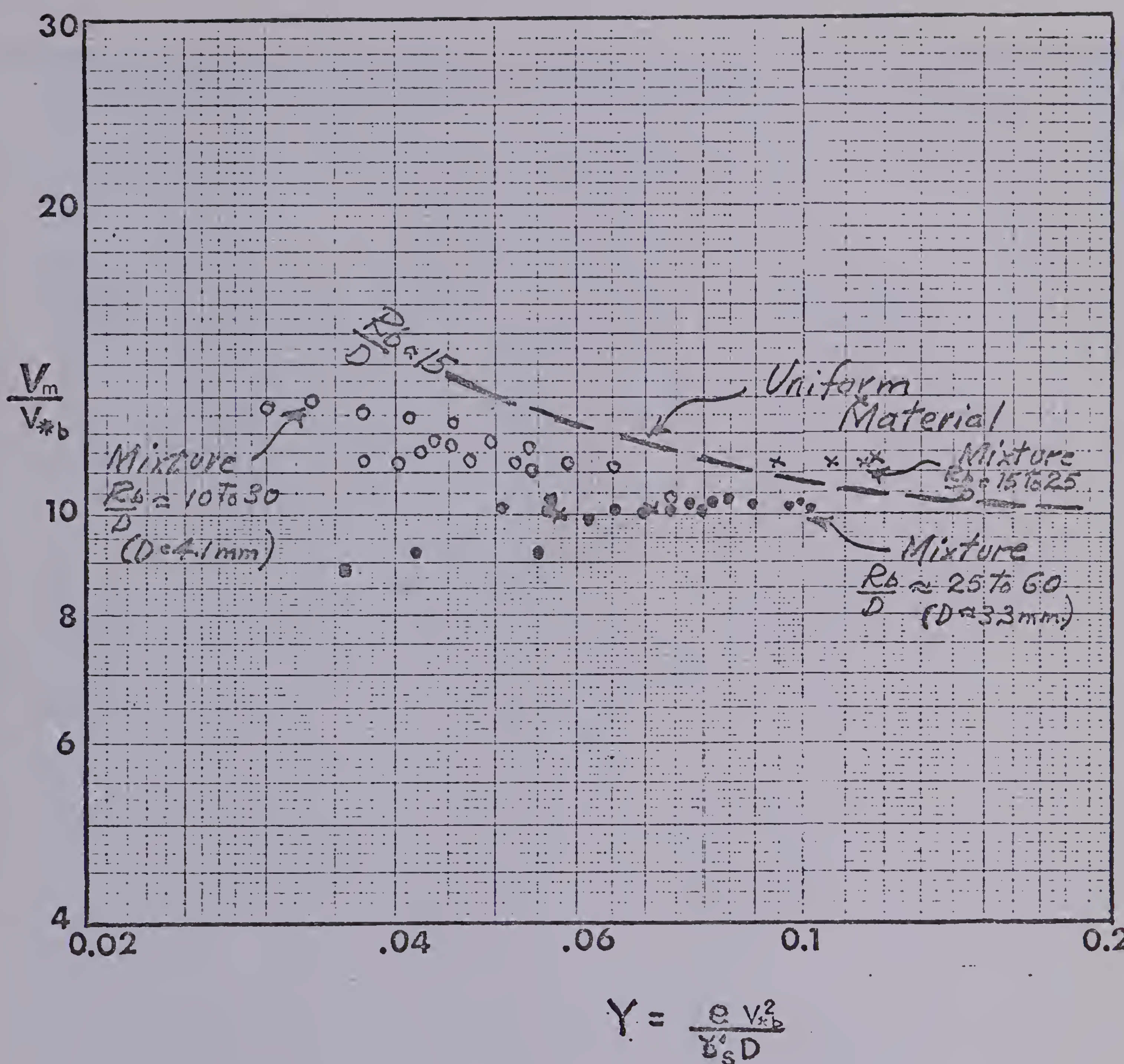
$\frac{V_m}{V_{*b}}$ VERSUS Y

FIGURE 88



$\frac{V_m}{V_{*b}}$ VERSUS Y

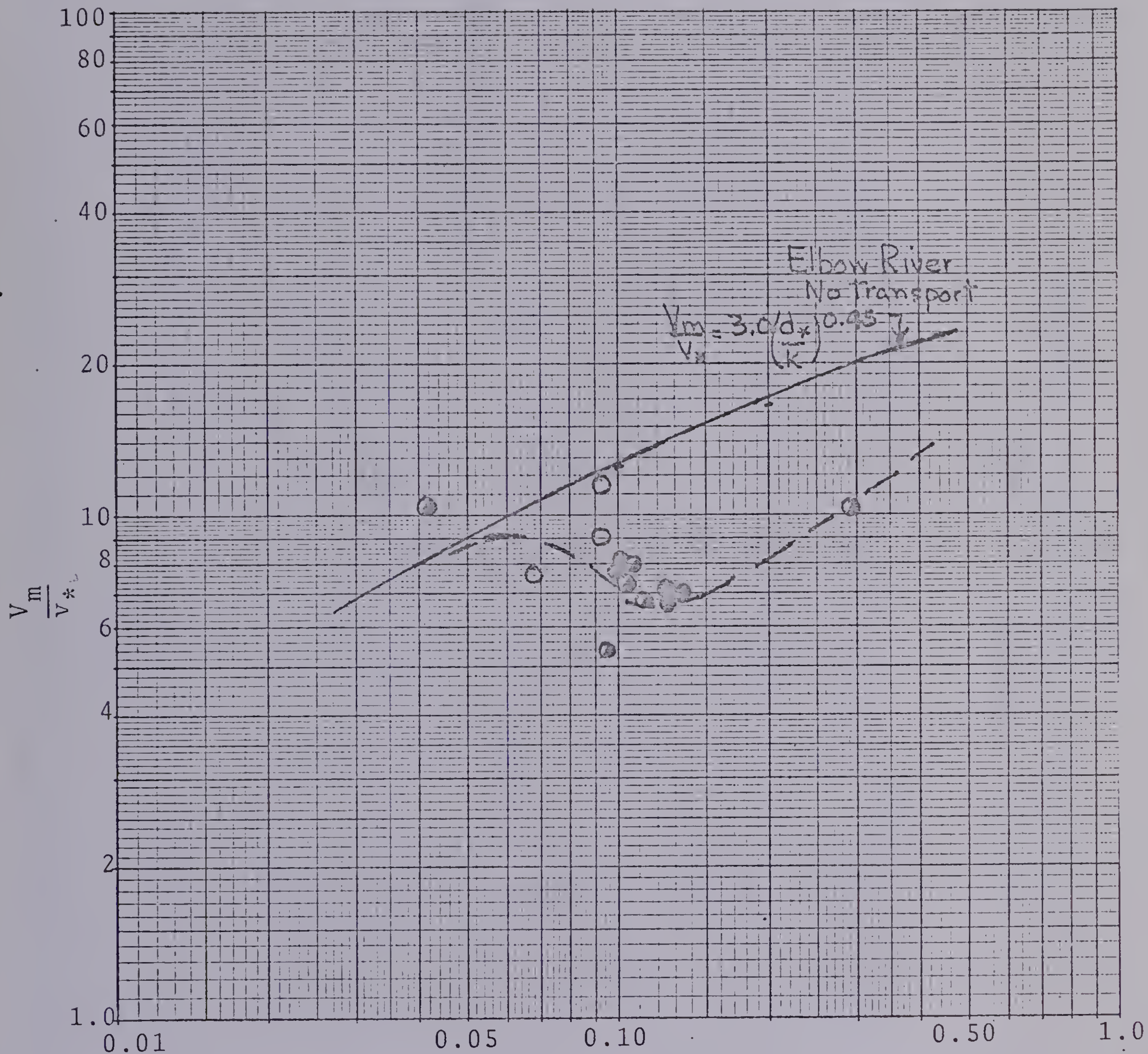
FIGURE 89



$\frac{V_m}{V_{*b}}$ VERSUS Y

COMPARISON BETWEEN UNIFORM MATERIALS AND MIXTURES

FIGURE 90

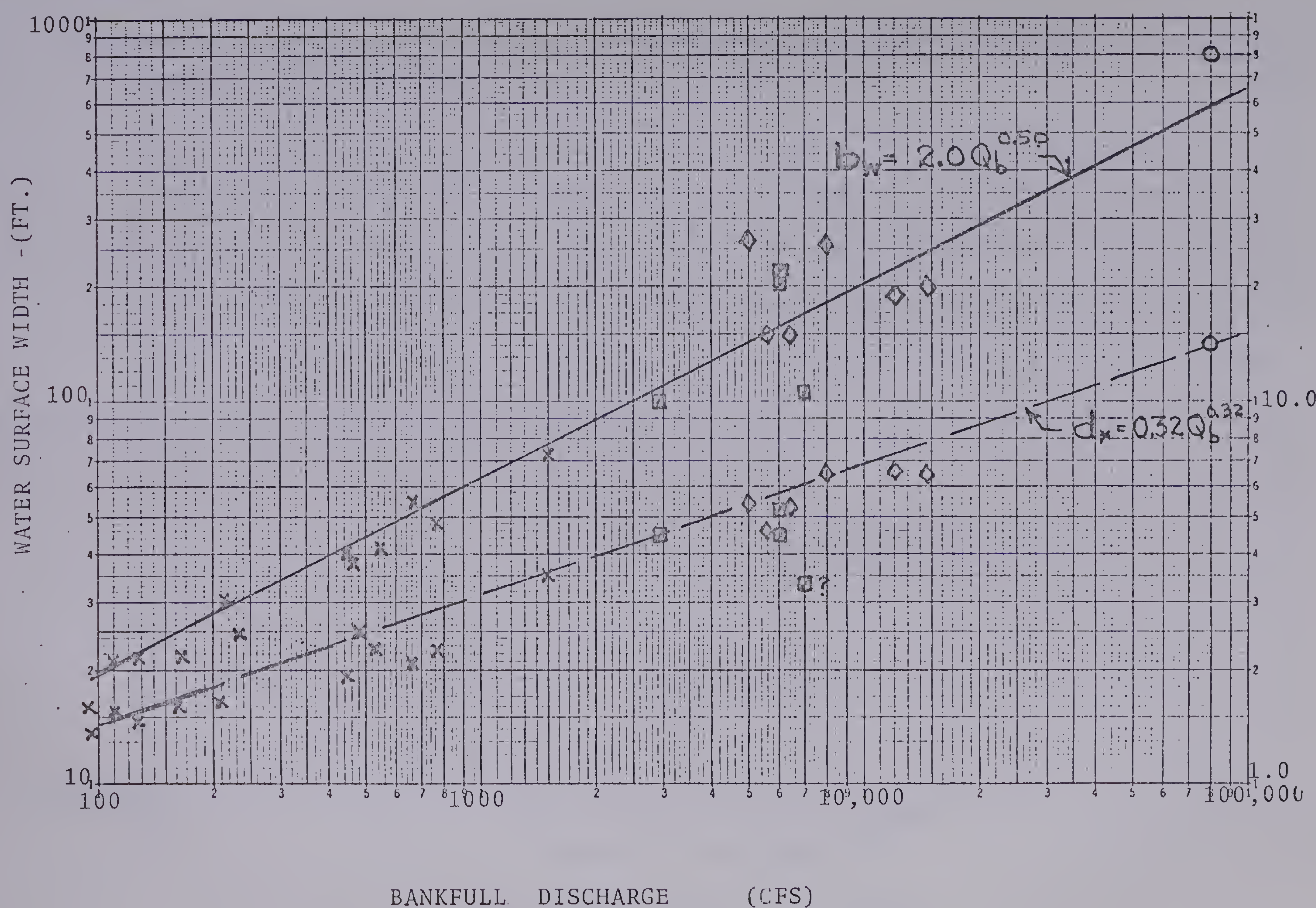


$$Y = \frac{e\, V_*^2}{\gamma'_s D}$$

NORTH SASKATCHEWAN RIVER- o
ELBOW RIVER- ●

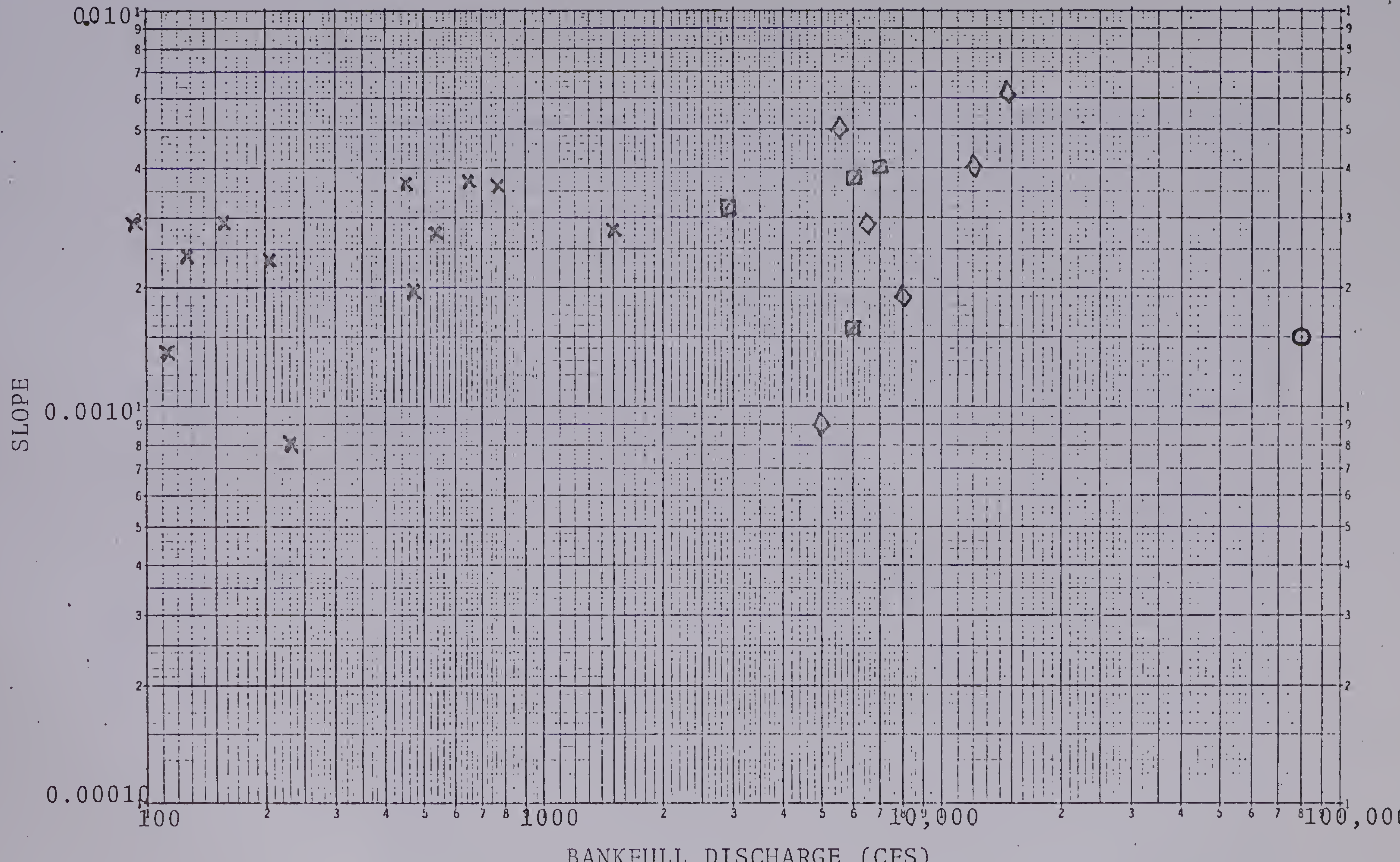
$\frac{V_m}{V_*}$ VERSUS Y

FIGURE 91



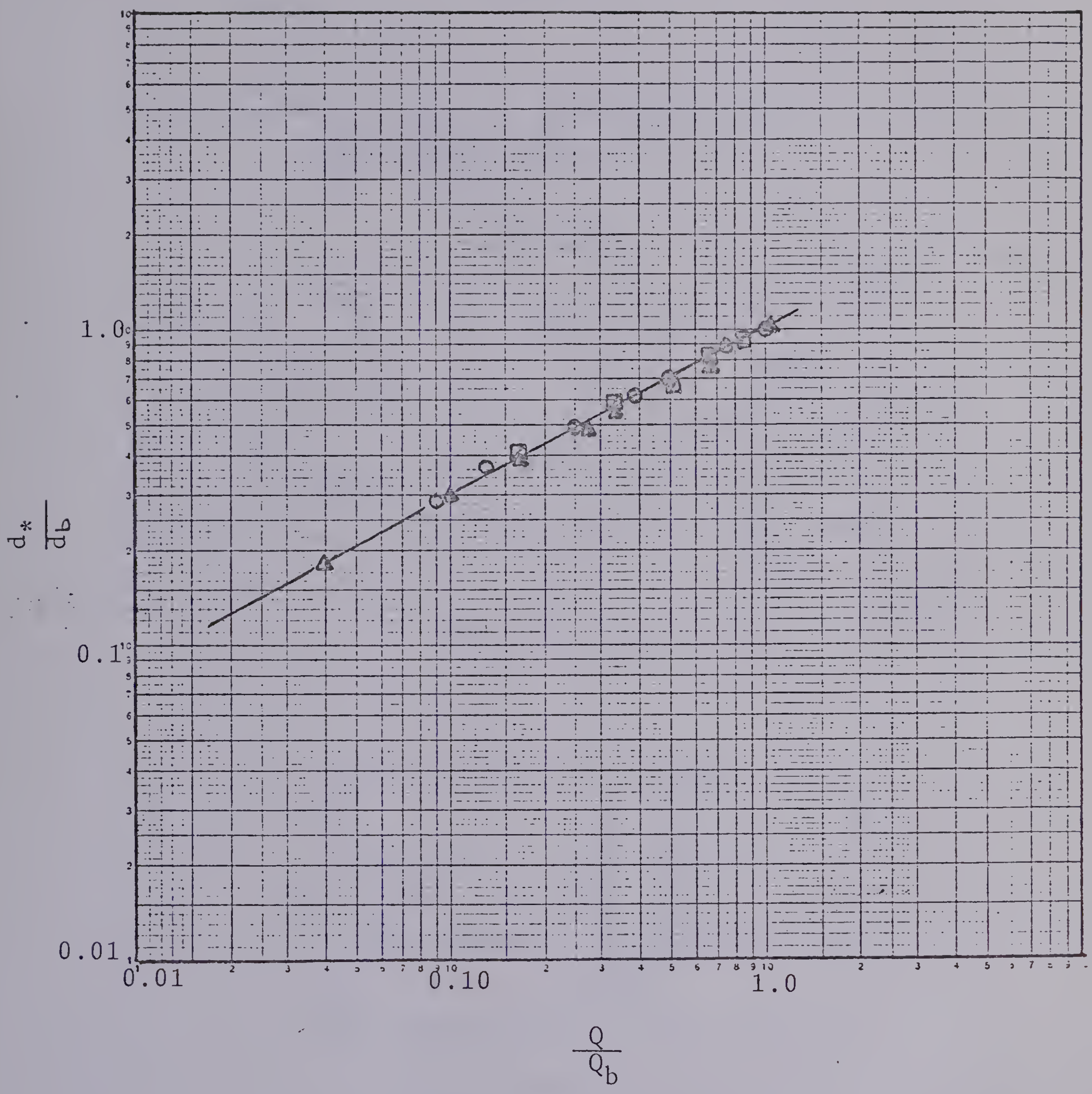
WIDTH AND DEPTH VERSUS BANKFULL DISCHARGE

FIGURE 92



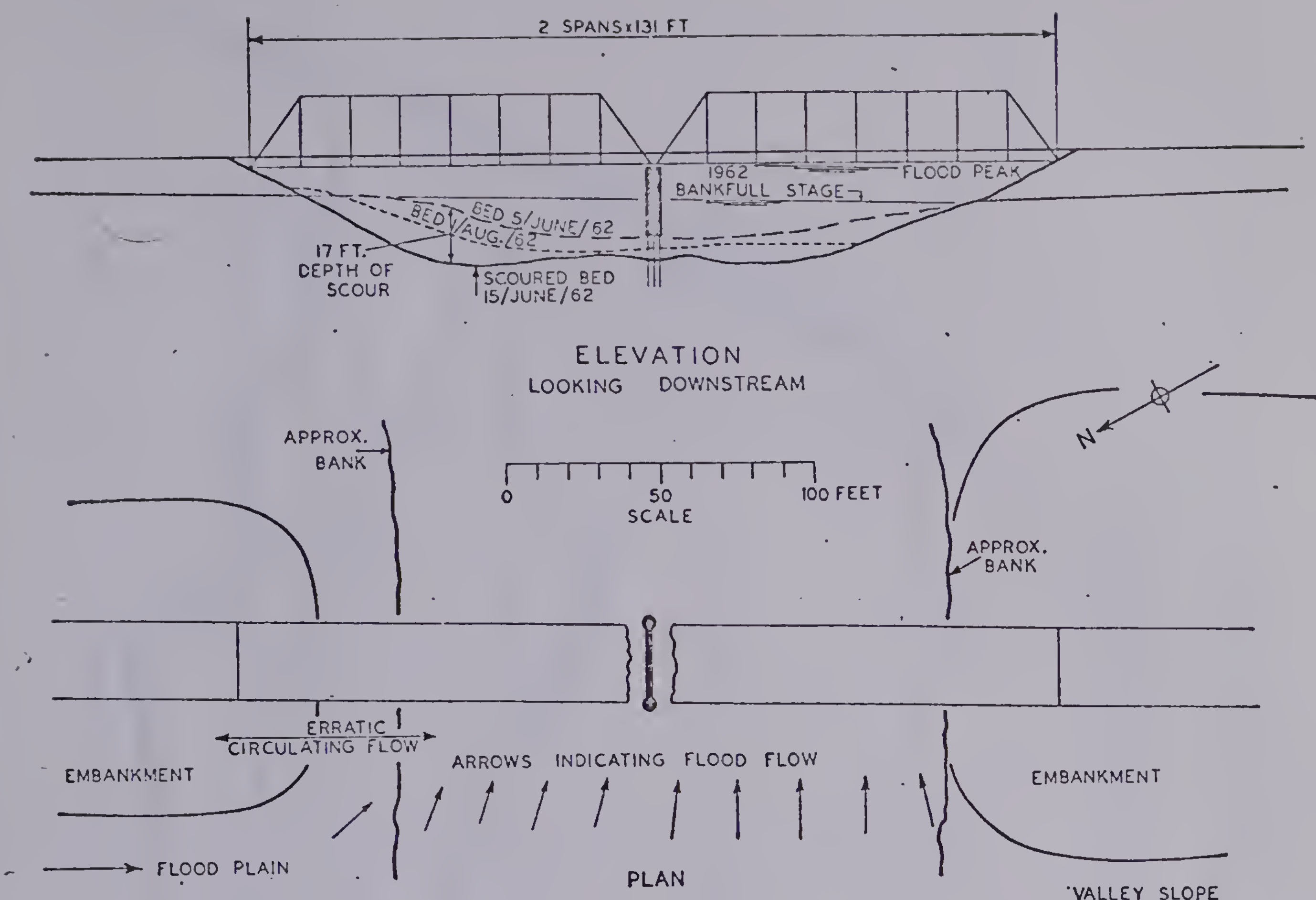
SLOPE VERSUS BANKFULL DISCHARGE

FIGURE 93

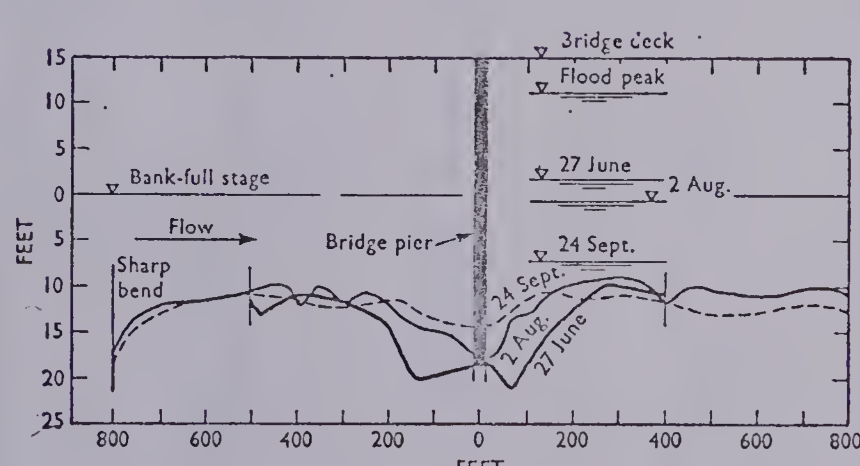


NON-DIMENSIONAL RATING CURVE
COARSE-BED CHANNELS

FIGURE 94



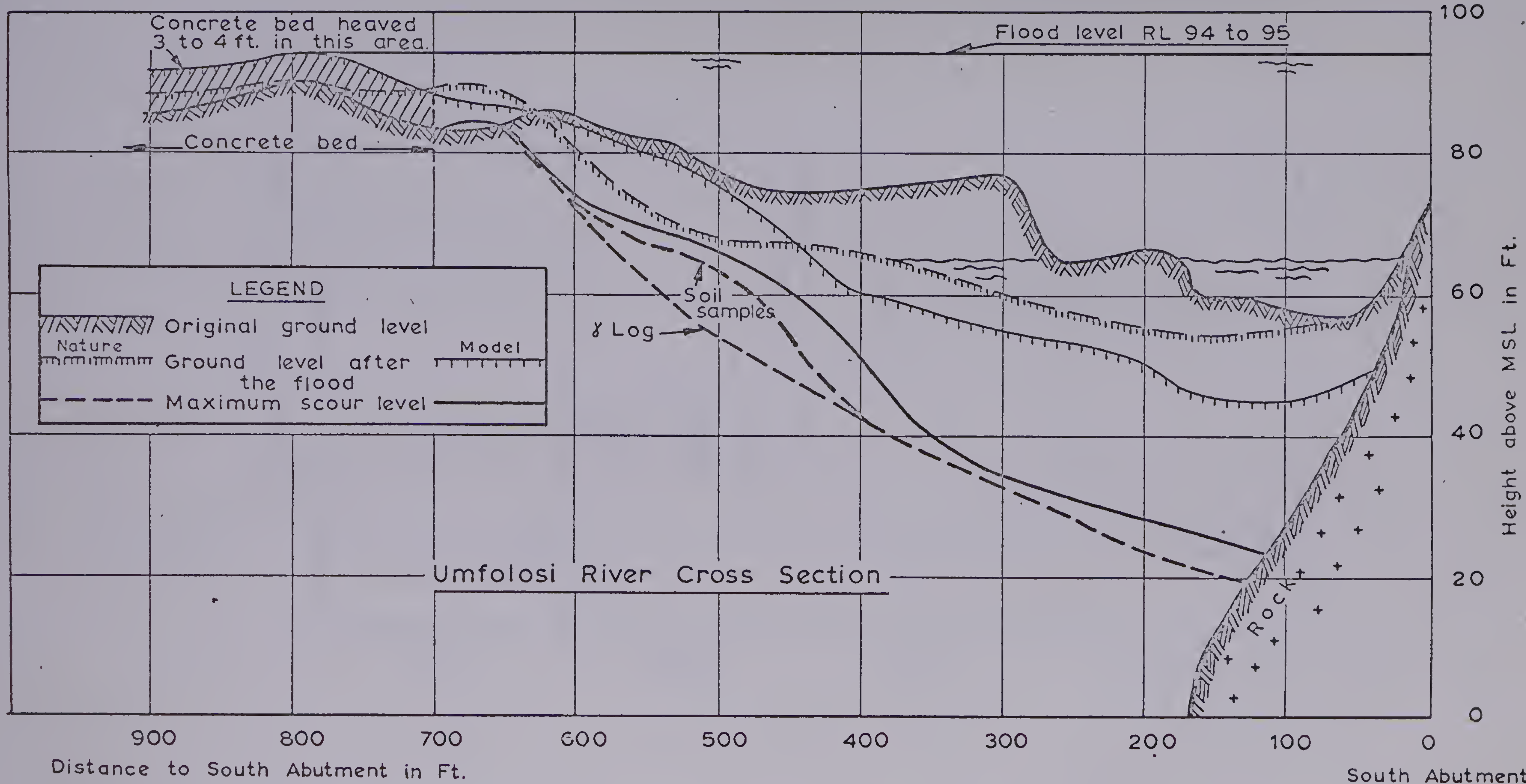
Details of Beaver Crossing bridge



Successive midstream profiles at Beaver Crossing

SCOUR AND FILL ON THE BEAVER
RIVER, ALBERTA
(After Neill, 1965)

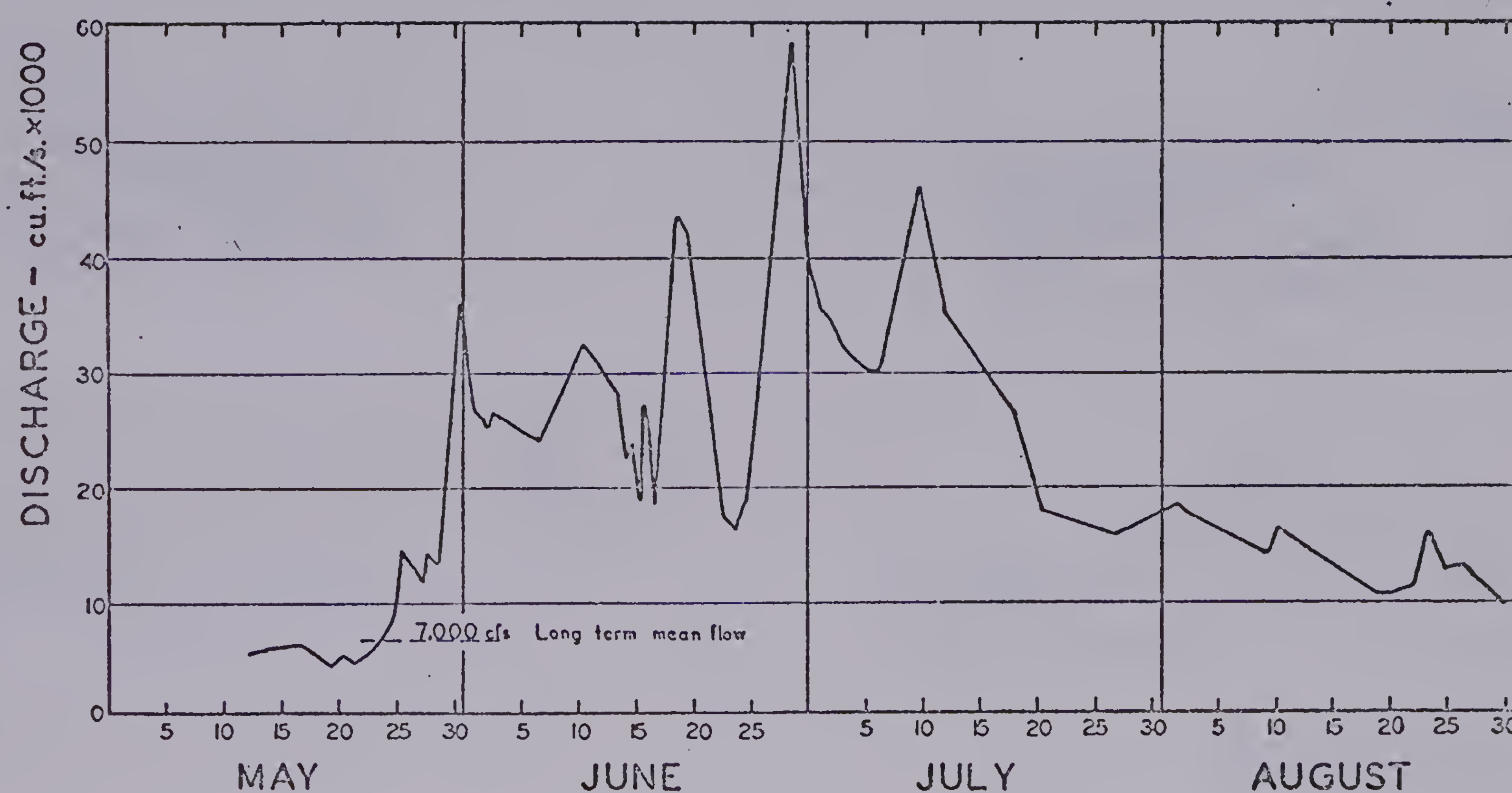
FIGURE 95



SCOUR DEPTHS AT BRIDGE SITE D.

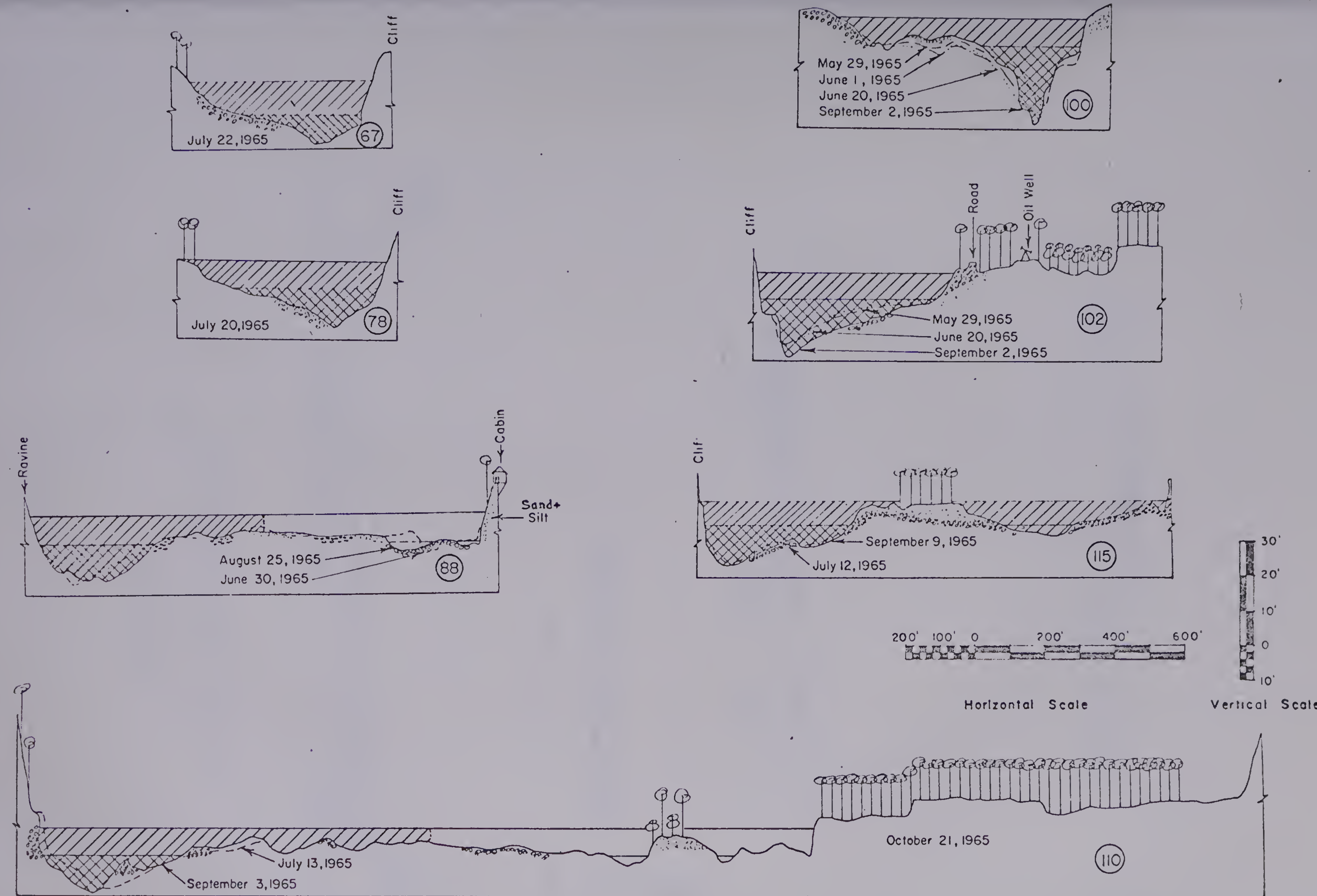
SCOUR AND FILL ON THE UMFOLOZI RIVER IN SOUTH AFRICA
(After Zwamborn, 1966)

FIGURE 96



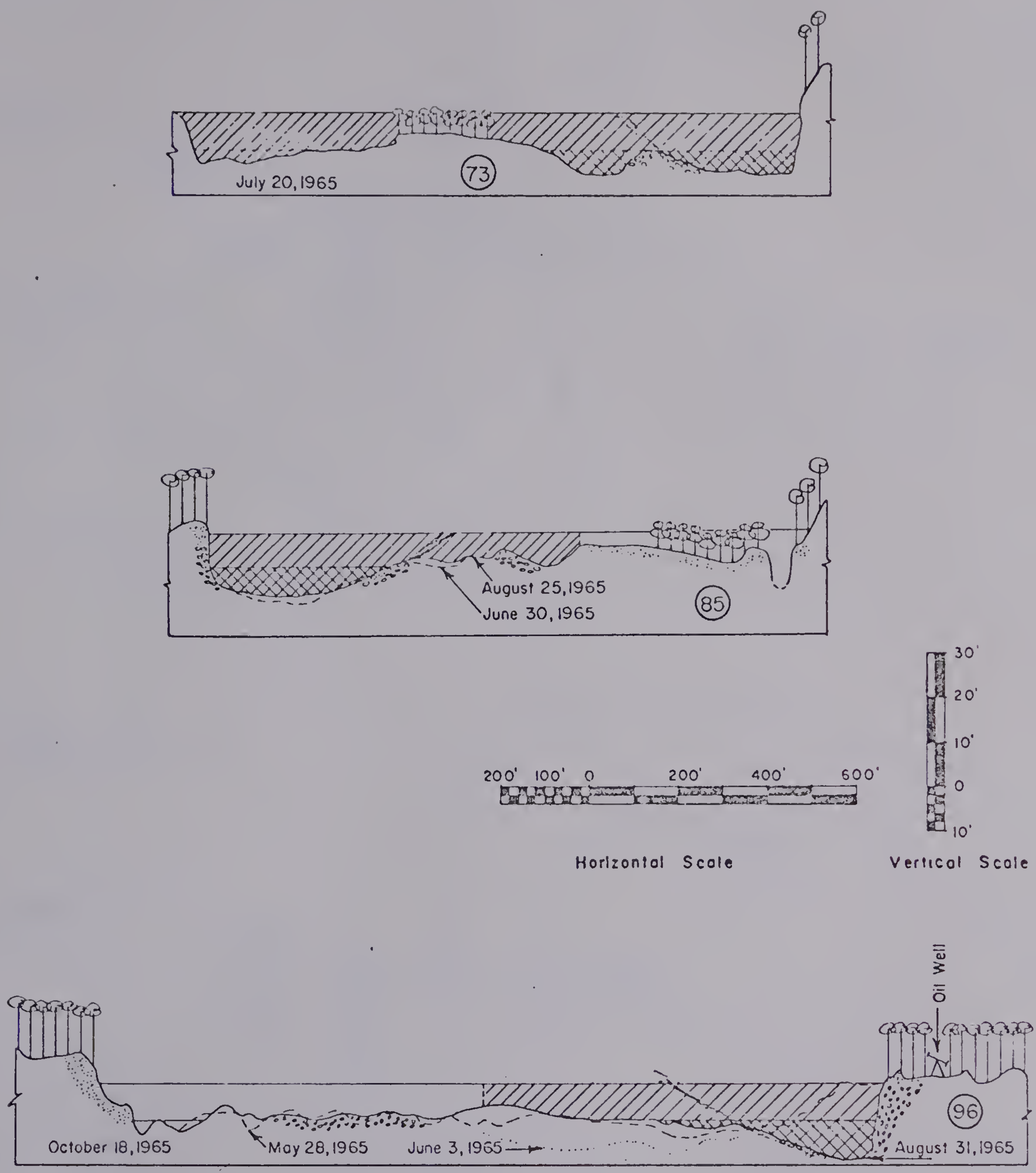
HYDROGRAPH FOR 1965
NORTH SASKATCHEWAN RIVER AT DRAYTON VALLEY

FIGURE 97



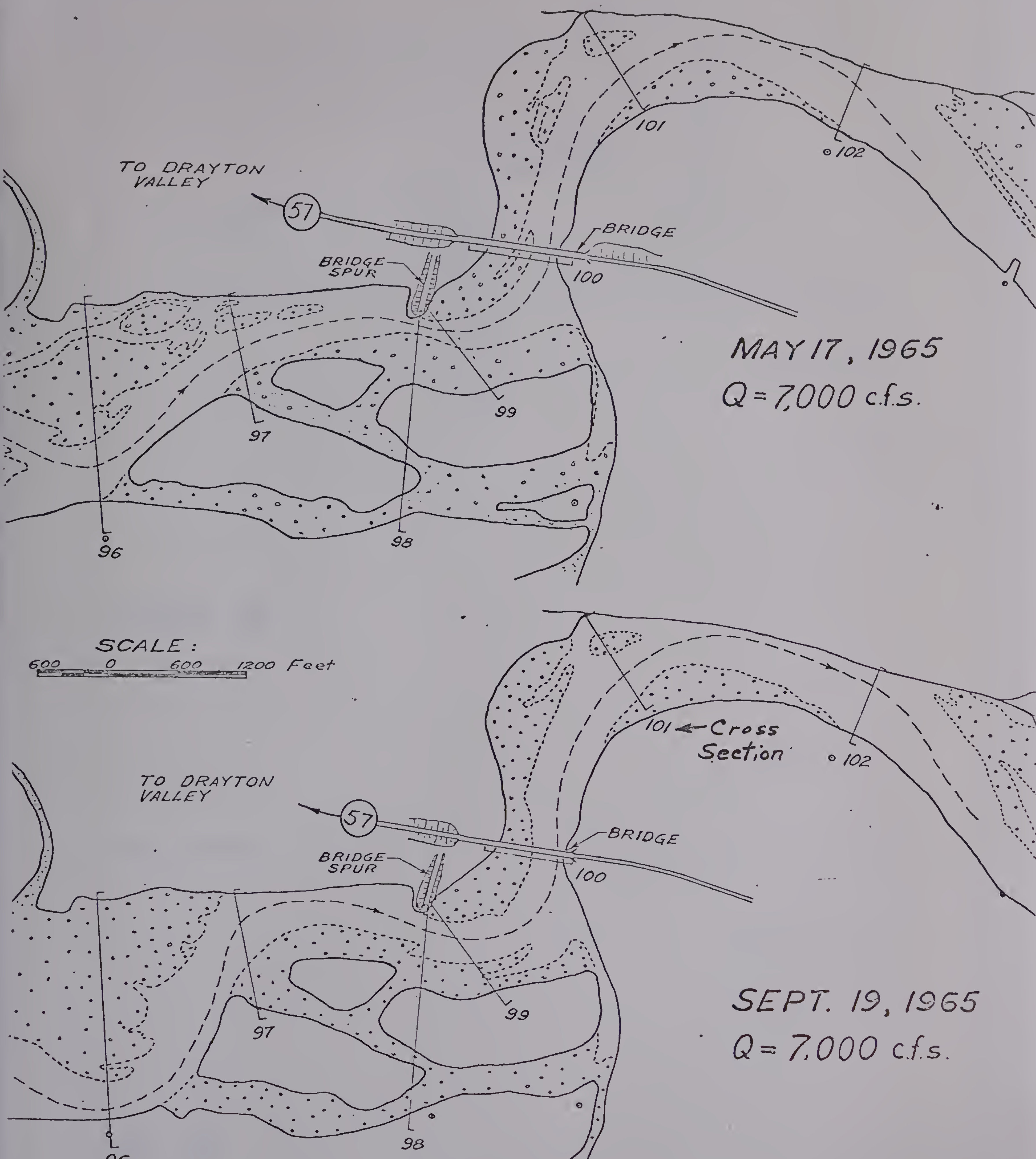
FORCED BEND CROSS-SECTIONS - NORTH SASKATCHEWAN RIVER

FIGURE 98



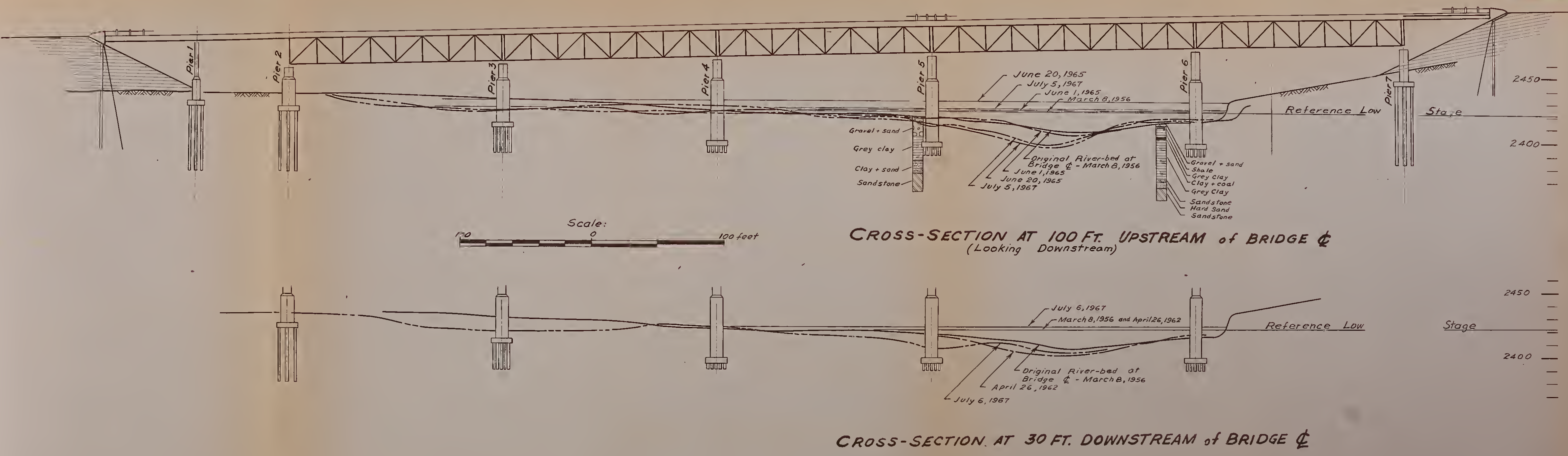
FREE BEND CROSS-SECTIONS
NORTH SASKATCHEWAN RIVER

FIGURE 99



CHANNEL SHIFT MAP
NORTH SASKATCHEWAN RIVER

FIGURE 100



SCOUR AT HIGHWAY NO. 57 BRIDGE

FIGURE 101

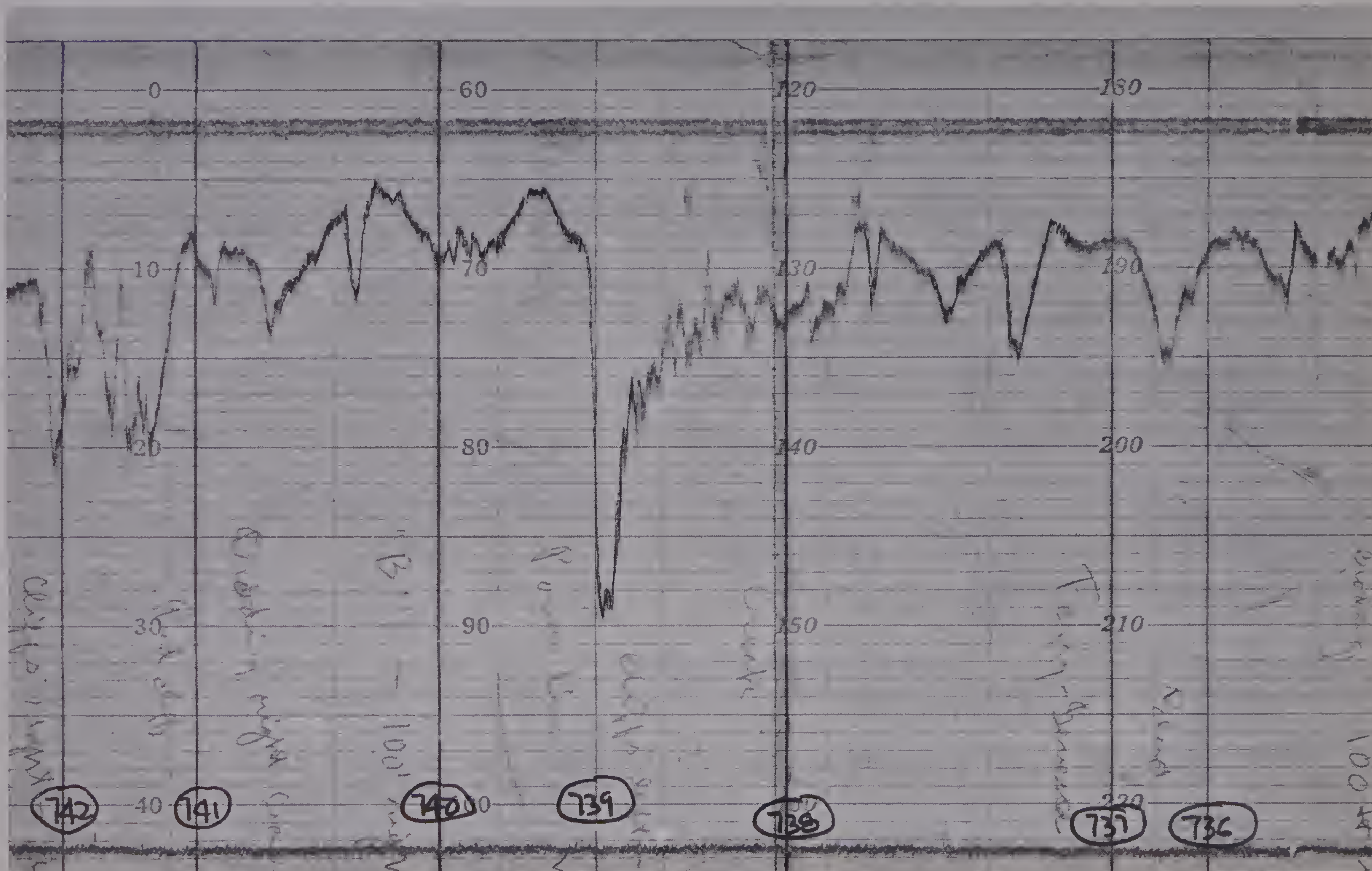
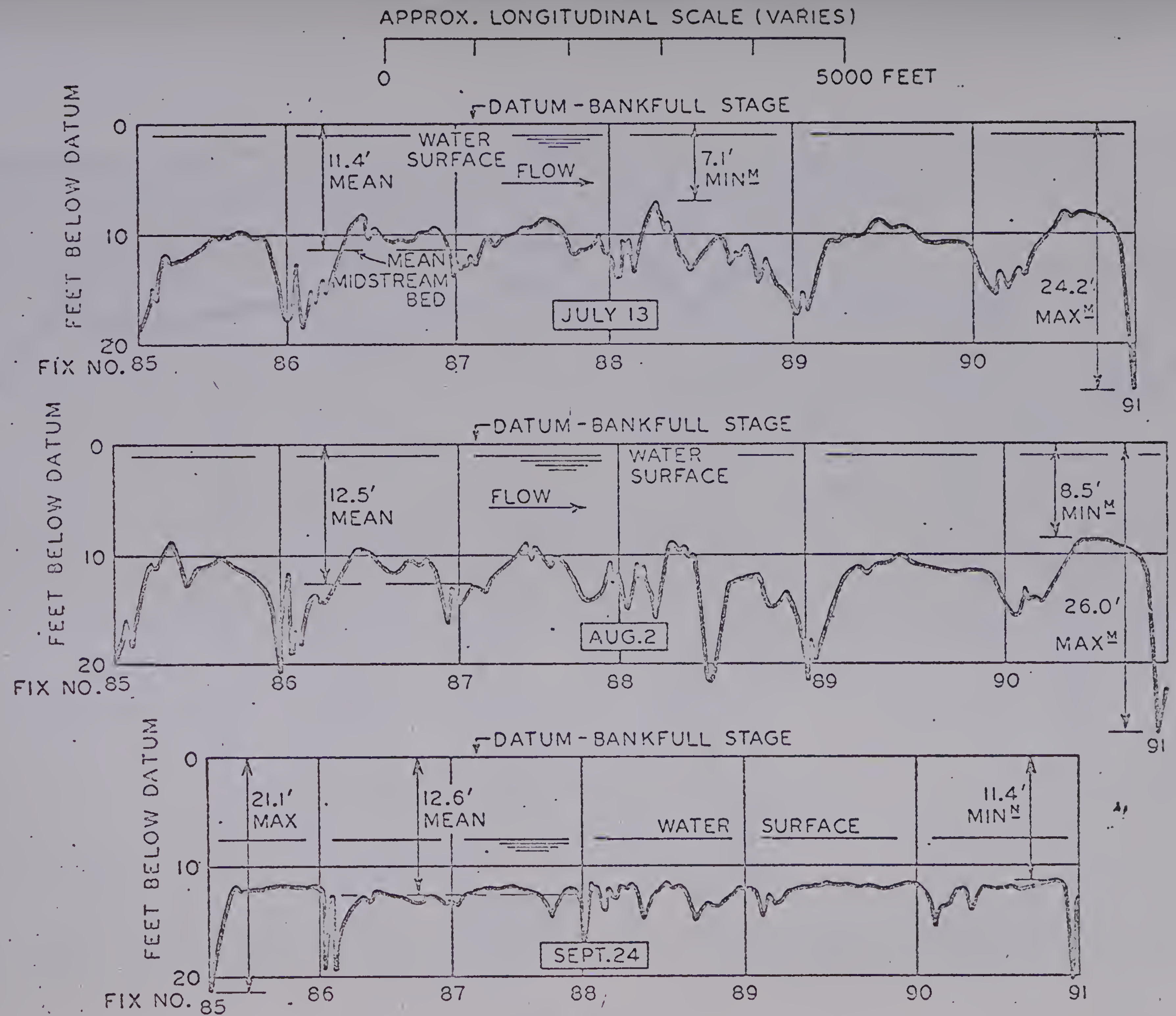


FIGURE 102
LONGITUDINAL SOUNDING ON THE NORTH SASKATCHEWAN RIVER NEAR GENESEE -(July 15,1966)



SUCCESSIVE MIDSTREAM PROFILES OF SAME REACH

BEAVER RIVER (After Neill, 1965)

FIGURE 103

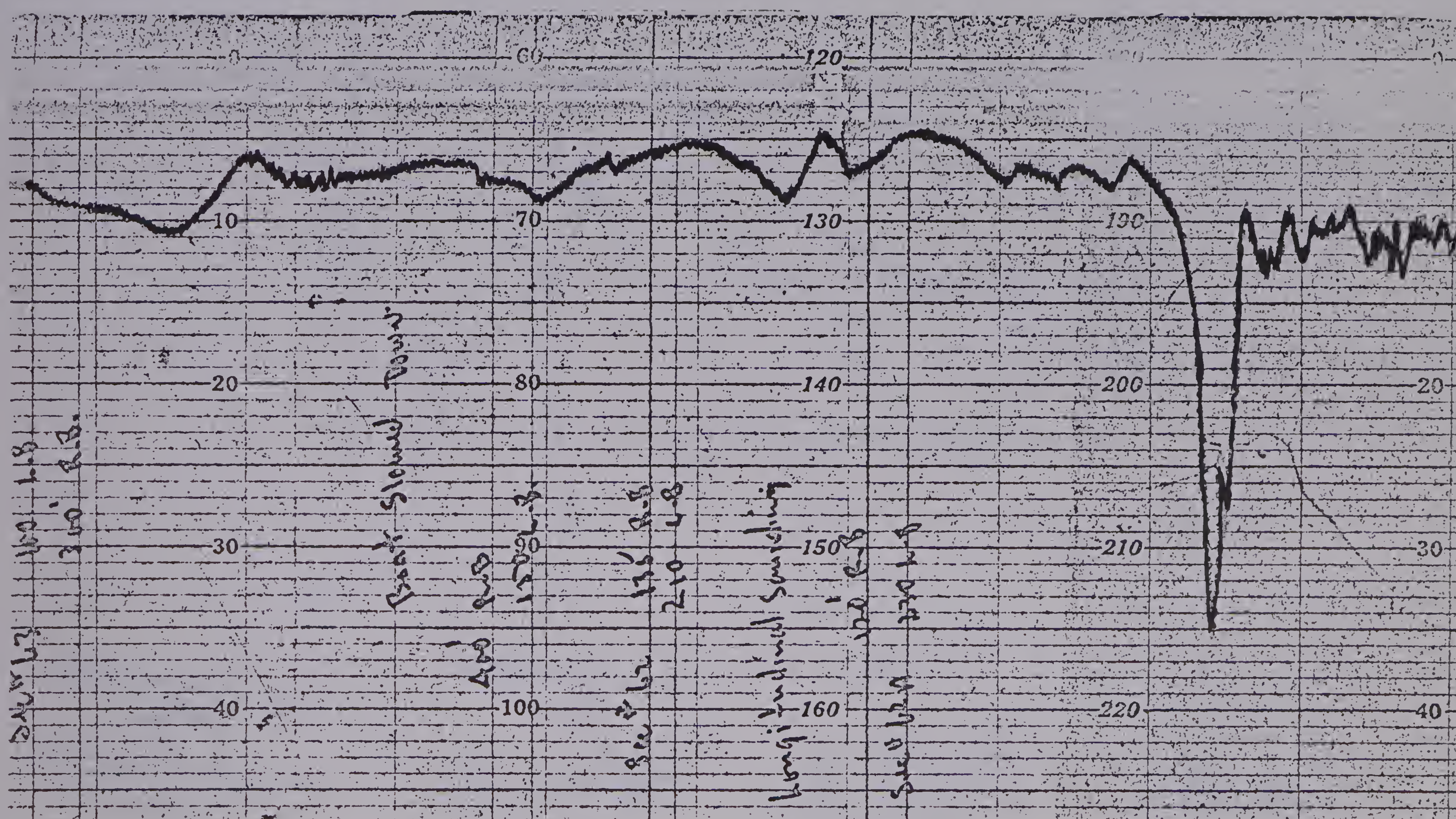
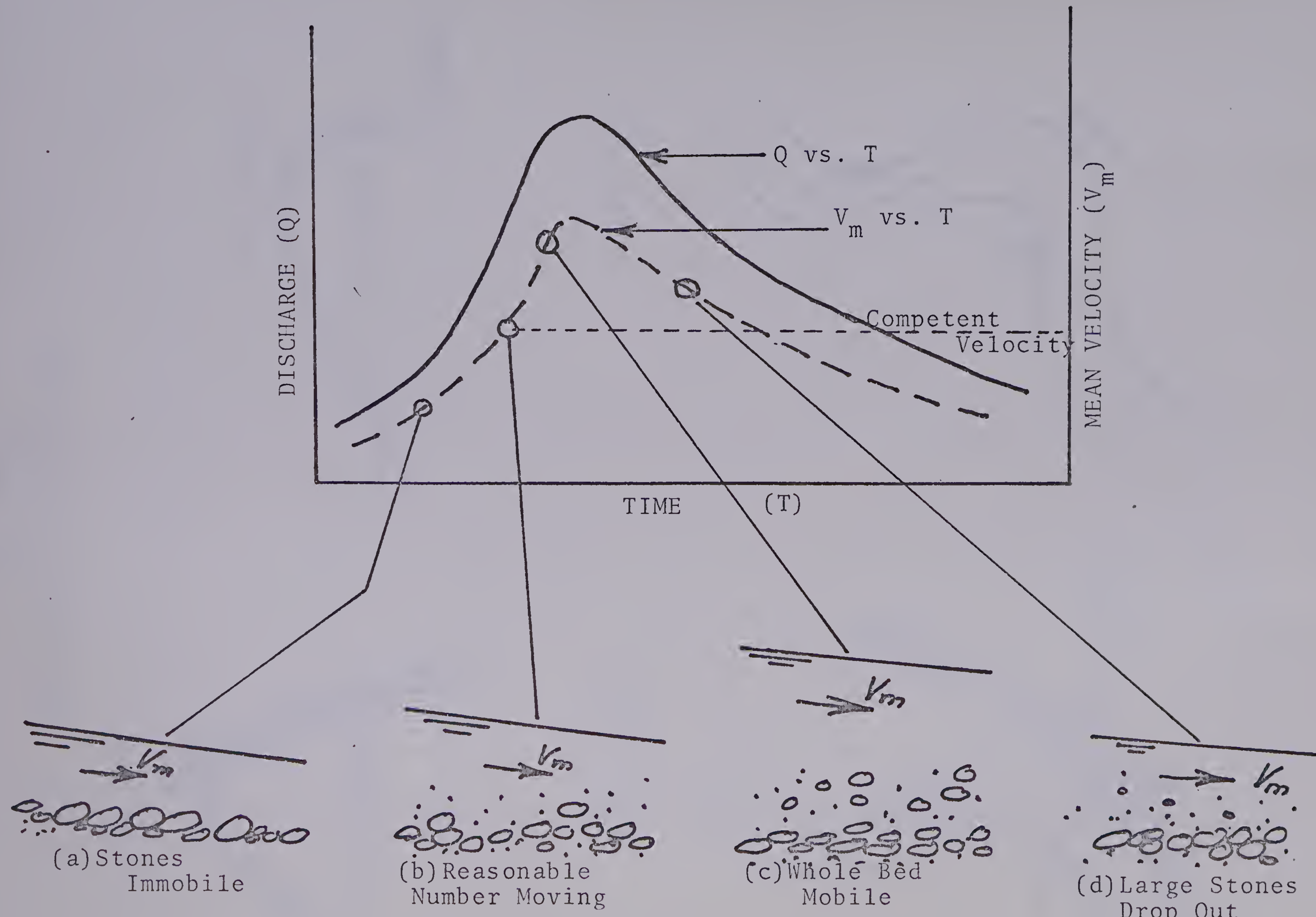
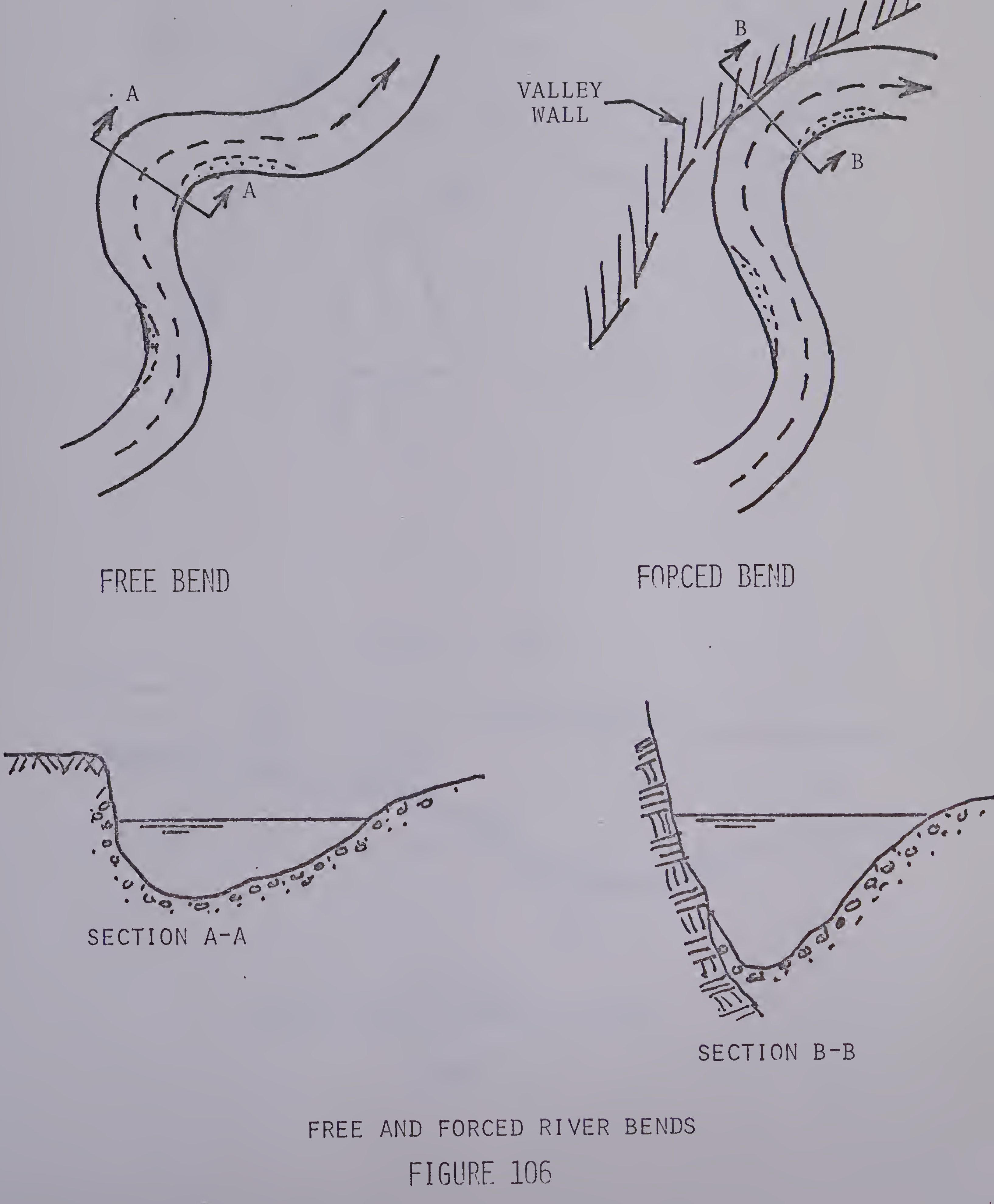


FIGURE 104
LONGITUDINAL THALWEG SOUNDING ON THE OLDMAN RIVER
UPSTREAM OF LETHBRIDGE -(June 13,1967)



BED ACTIVITY DURING PASSAGE OF FLOOD (HYPOTHETICAL)

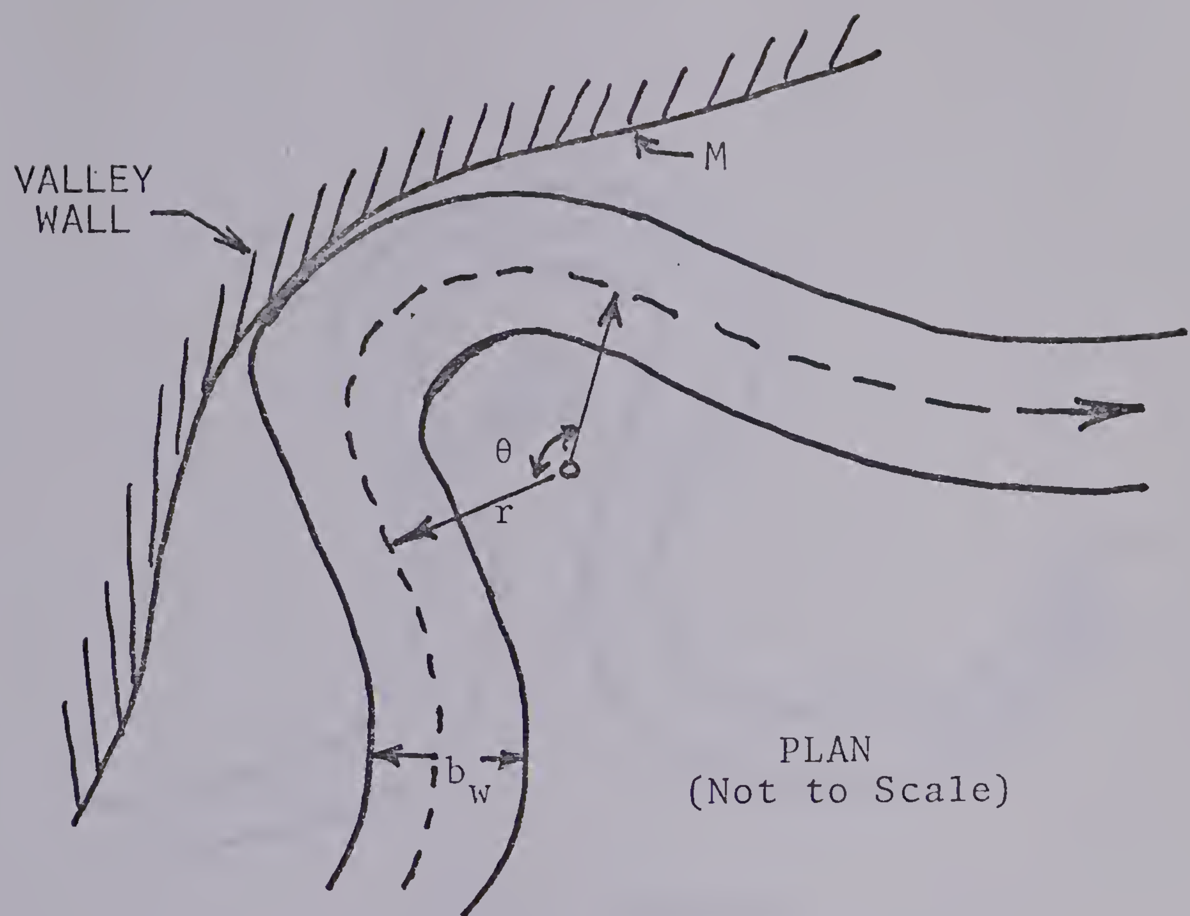
FIGURE 105



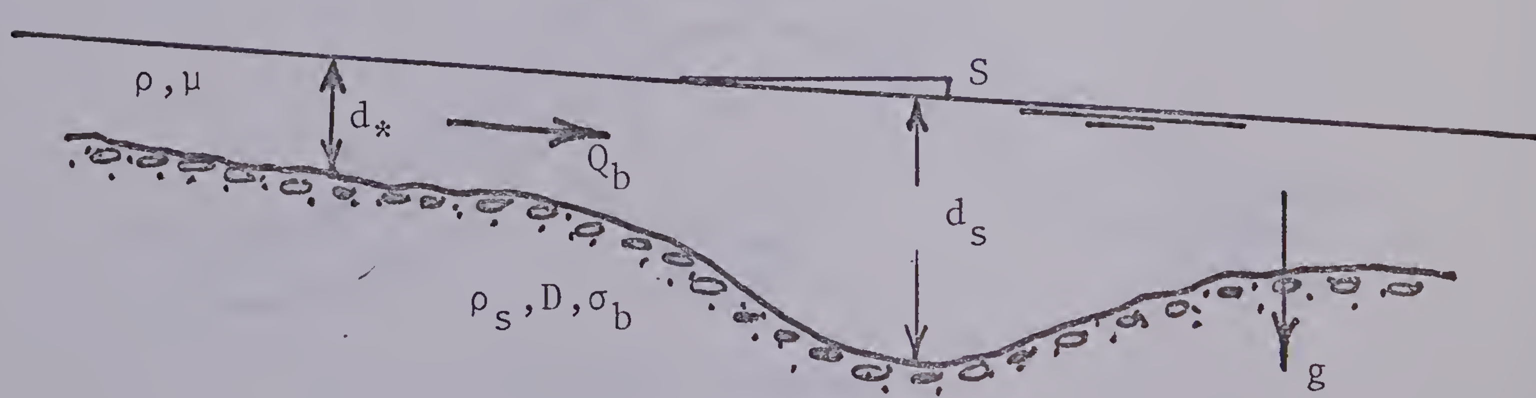
FREE AND FORCED RIVER BENDS

FIGURE 106

FORCED BEND



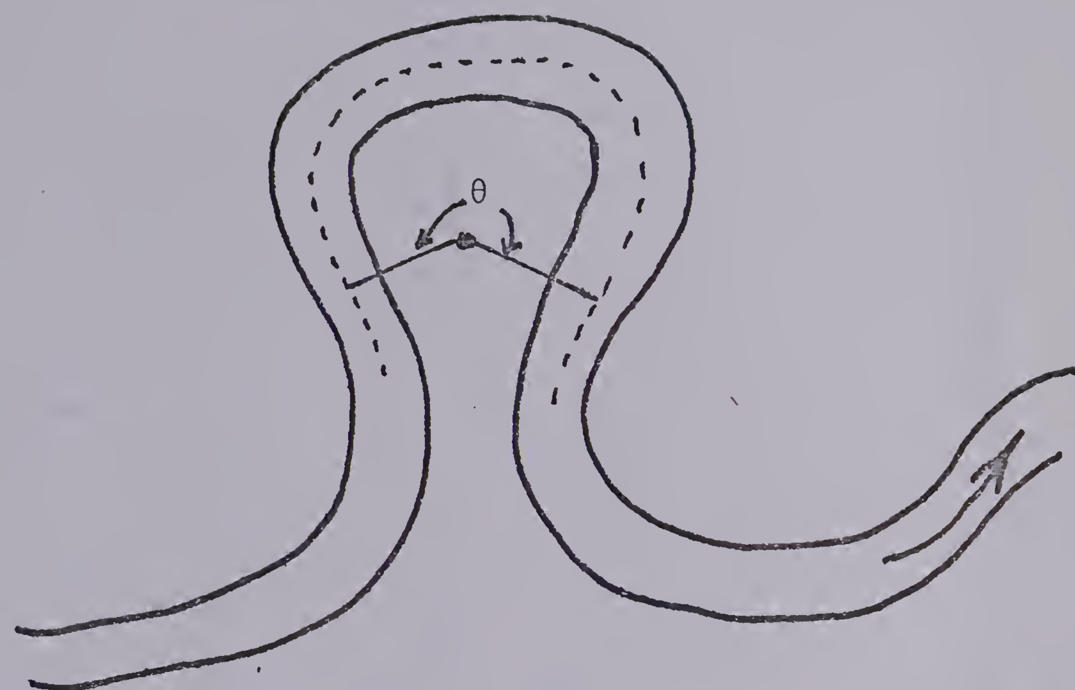
BANKFULL STAGE



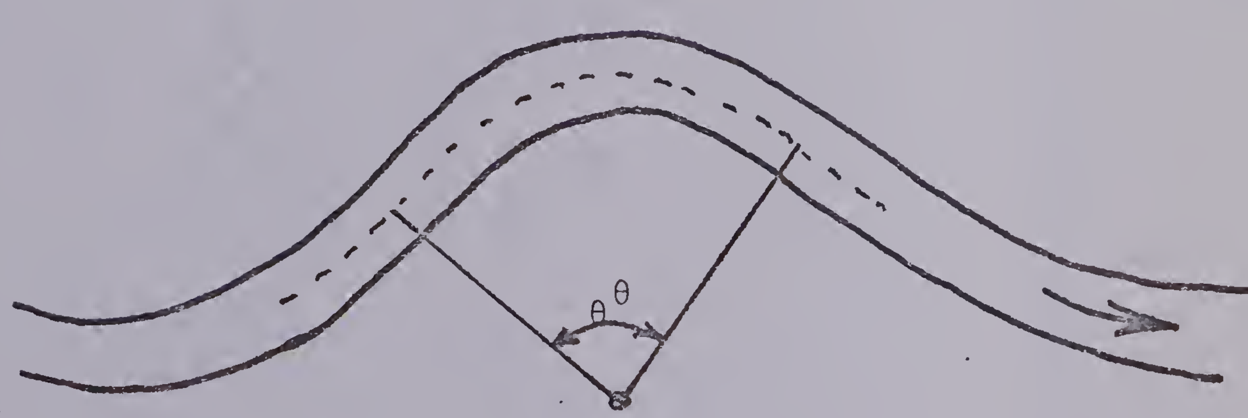
PROFILE

VARIABLES
FORCED BEND AT BANKFULL STAGE

FIGURE 107



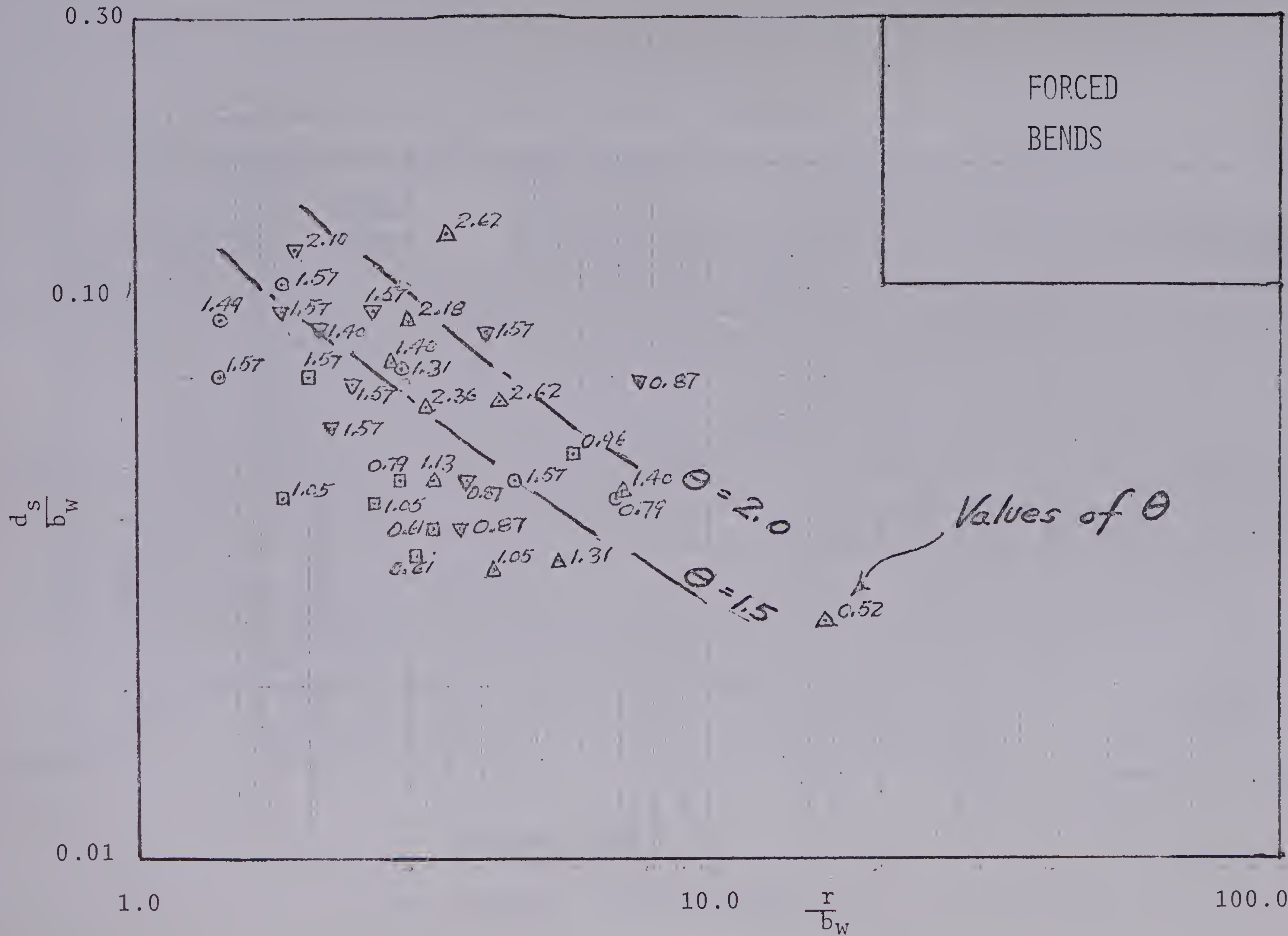
(a) SHARP MEANDER



(b) GRADUAL MEANDER

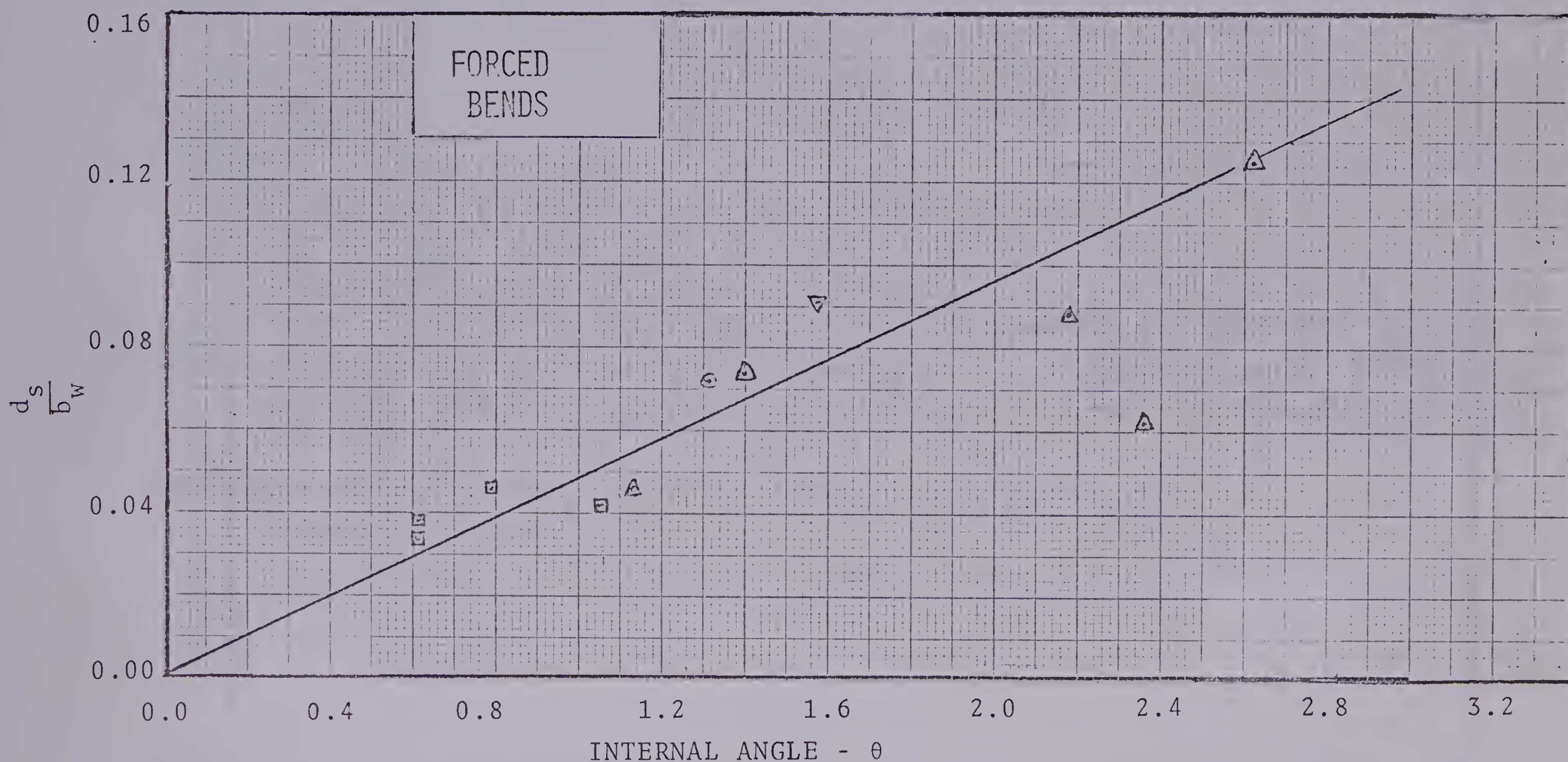
COMPARISON OF BENDS HAVING DIFFERENT
INTERNAL ANGLES OF CURVATURE

FIGURE 108



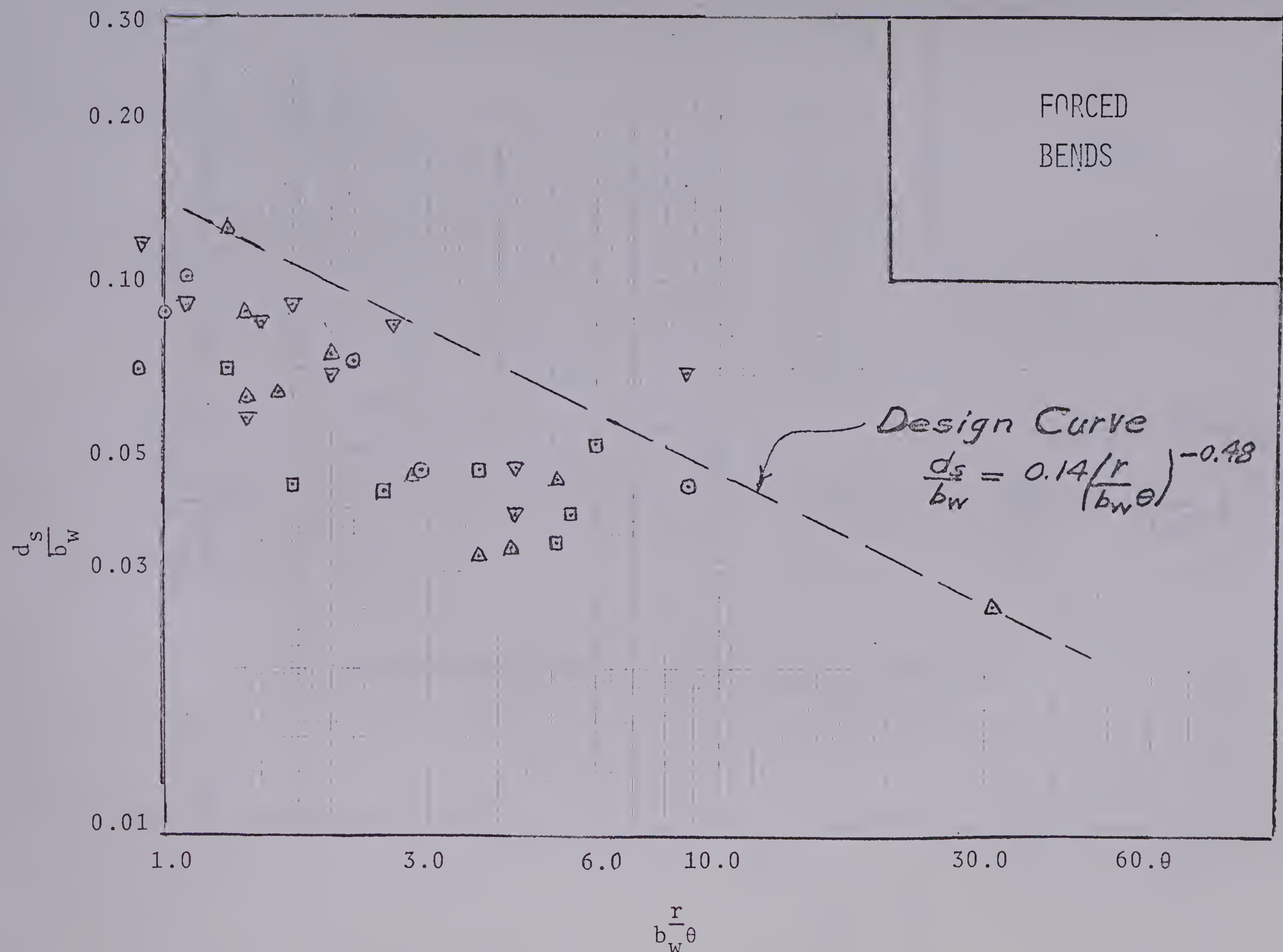
$\frac{d_s}{b_w}$ VERSUS $\frac{r}{b_w}$ FOR VARYING INTERNAL ANGLE

FIGURE 109



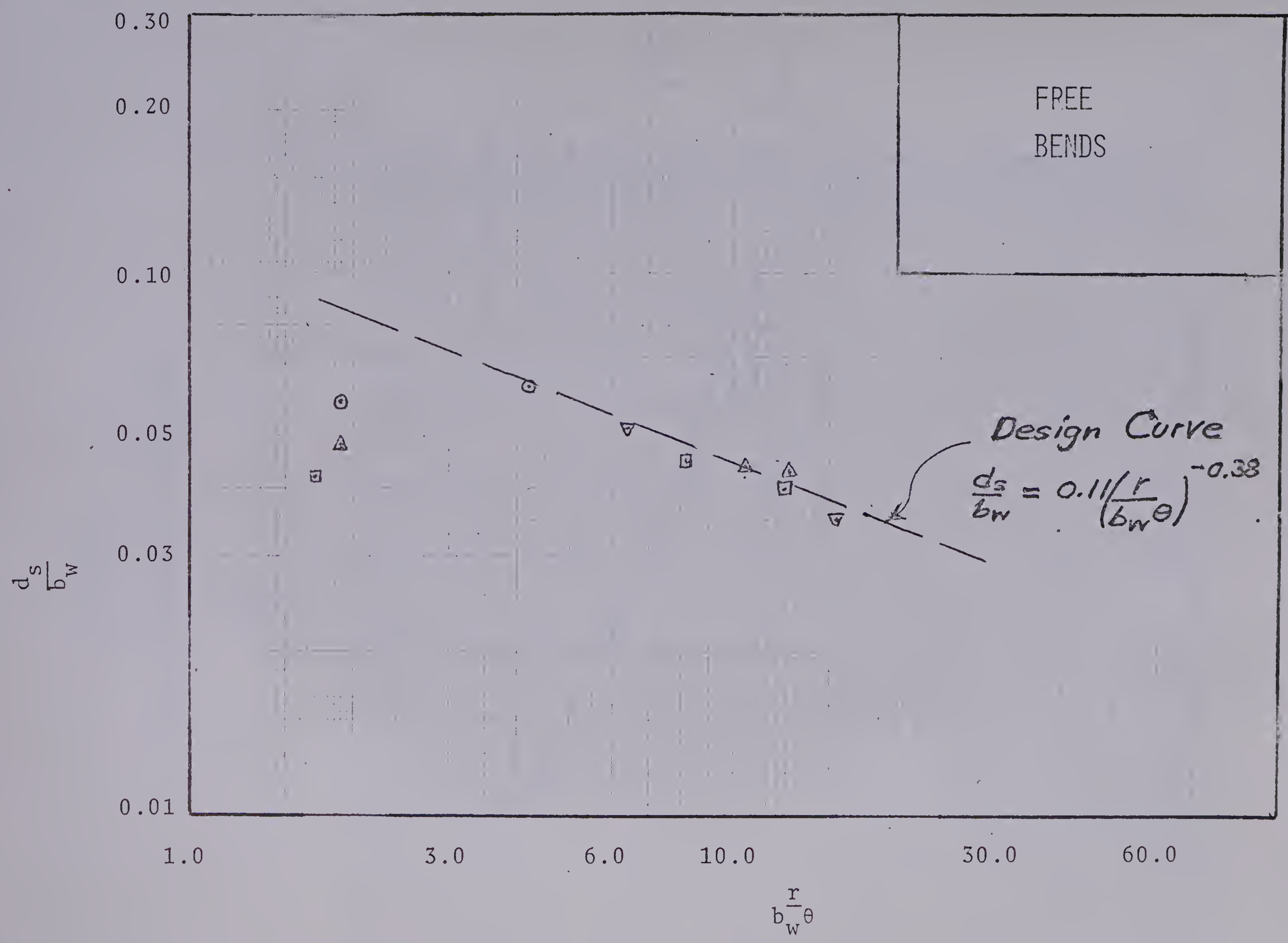
$\frac{d_s}{b_w}$ VERSUS INTERNAL ANGLE FOR $\frac{r}{b_w}$ = 2.5 to 3.5

FIGURE 110



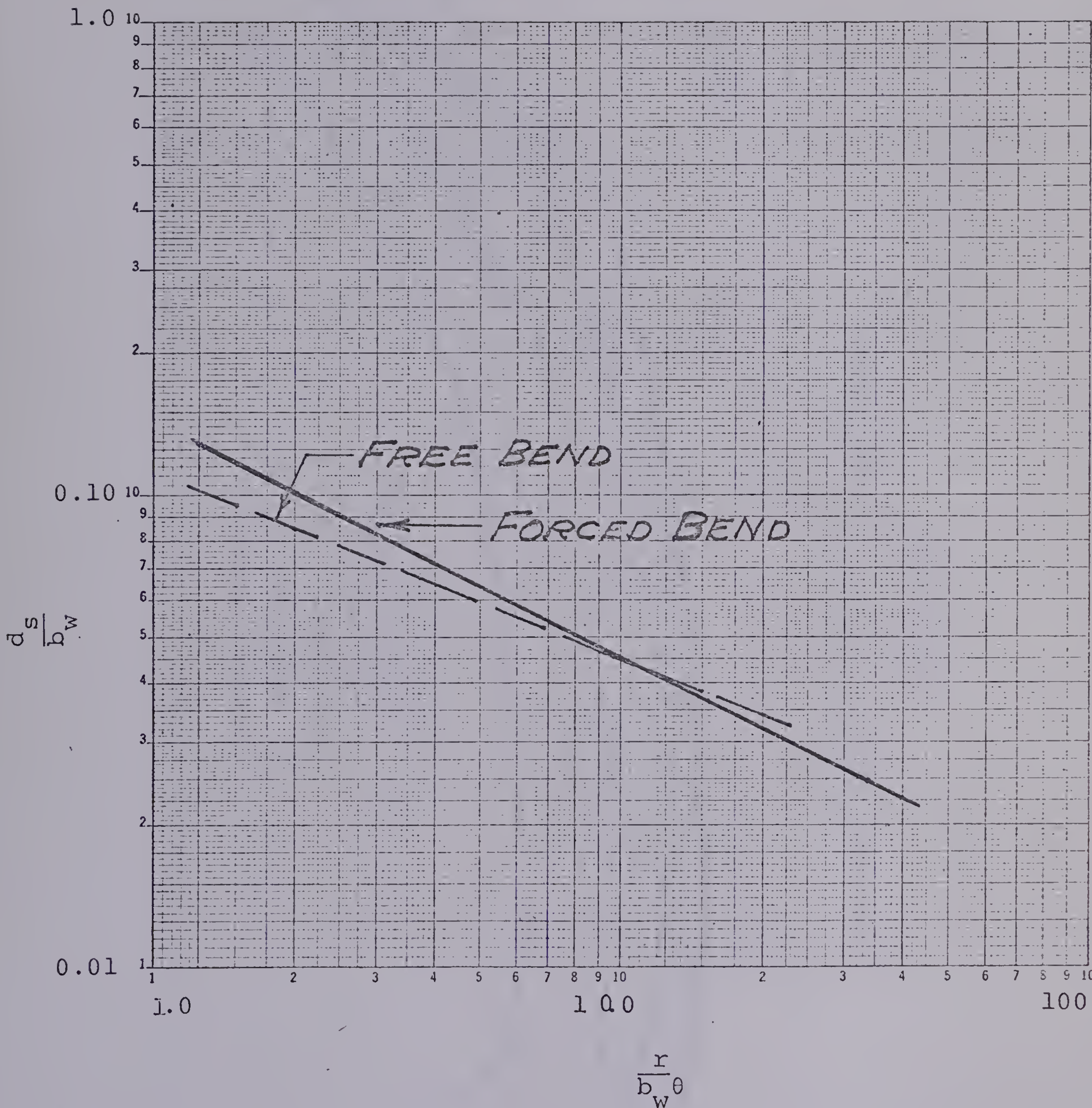
$\frac{d_s}{b_w}$ VERSUS $\frac{r}{b_w \theta}$

FIGURE 111



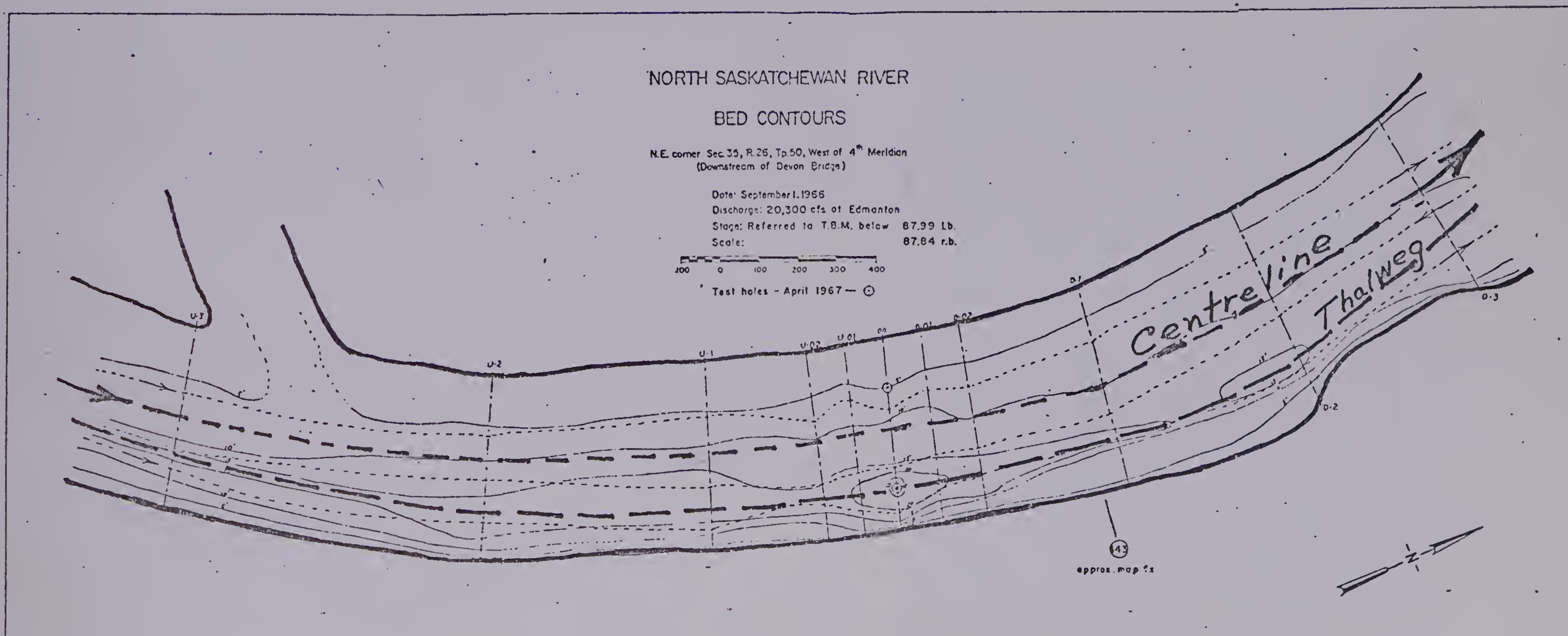
$\frac{d_s}{b_w}$ VERSUS $\frac{r}{b_w\theta}$

FIGURE 112



COMPARISON OF DESIGN CURVES FOR
FREE AND FORCED BENDS

FIGURE 113



RIVER BEND SHOWING CENTRELINE AND THALWEG

FIGURE 114

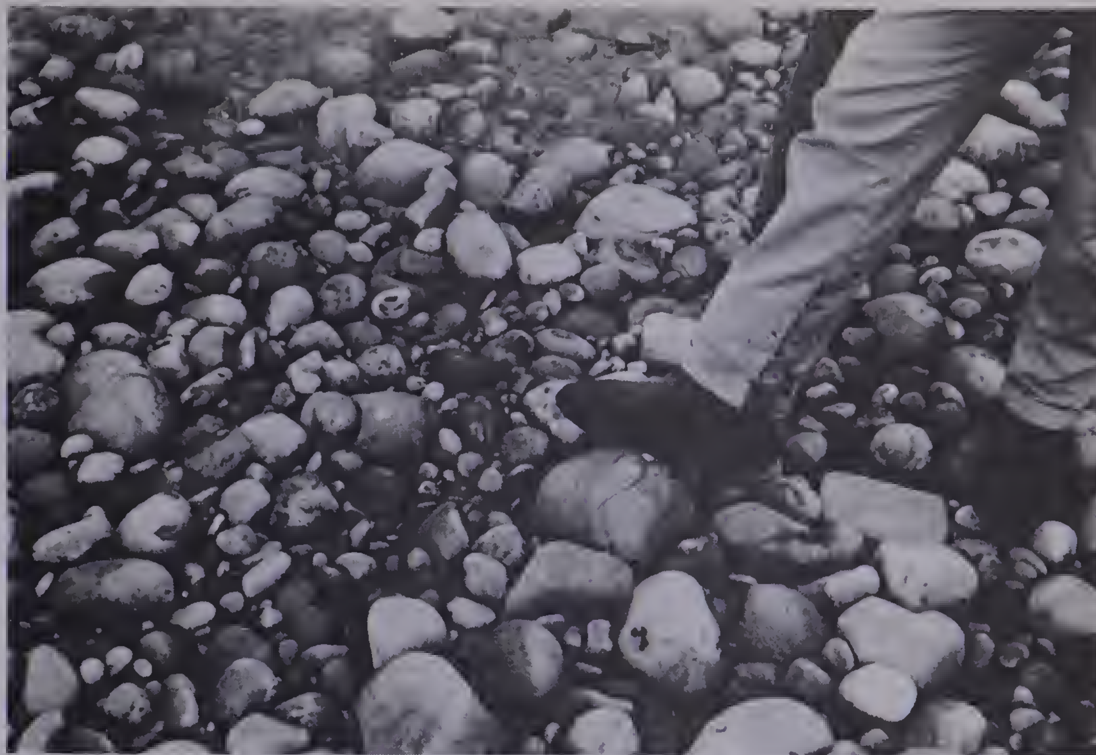
PHOTOGRAPHS

PHOTOGRAPH 1

SCOOP SAMPLE BEING OBTAINED AT 6 INCHES BELOW BAR SURFACE.
(North Sask. River, Drayton Valley)
x-sec 100

PHOTOGRAPH 2

SQUARE-DEPTH SAMPLING TECHNIQUE
(Lee Creek at Cardston)



PHOTOGRAPH 3

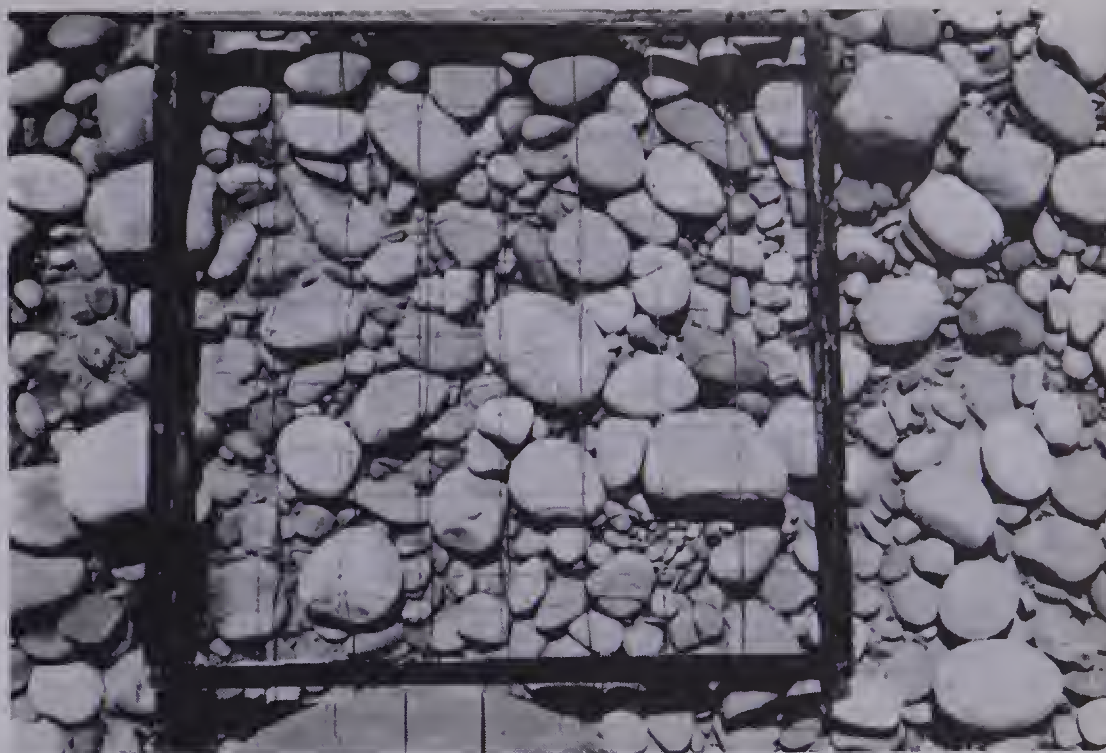
GRID SAMPLING TECHNIQUE BY PACING

(North Sask. River, Drayton Valley)
x-sec 101

PHOTOGRAPH 4

LINE SAMPLING TECHNIQUE ALONG A RIVER BANK

(North Sask. River, Drayton Valley)



PHOTOGRAPH 5

PHOTOGRAPHIC SQUARE-SURFACE SAMPLE
SQUARE IS 2 FT. x 2 FT.

(North Sask. River, Drayton Valley)
x-sec 89

PHOTOGRAPH 6

THRESHOLD OF MOTION STUDIES
STONES PLACED IN SEPARATE SQUARES
(North Sask. River, Drayton Valley)
x-sec 89

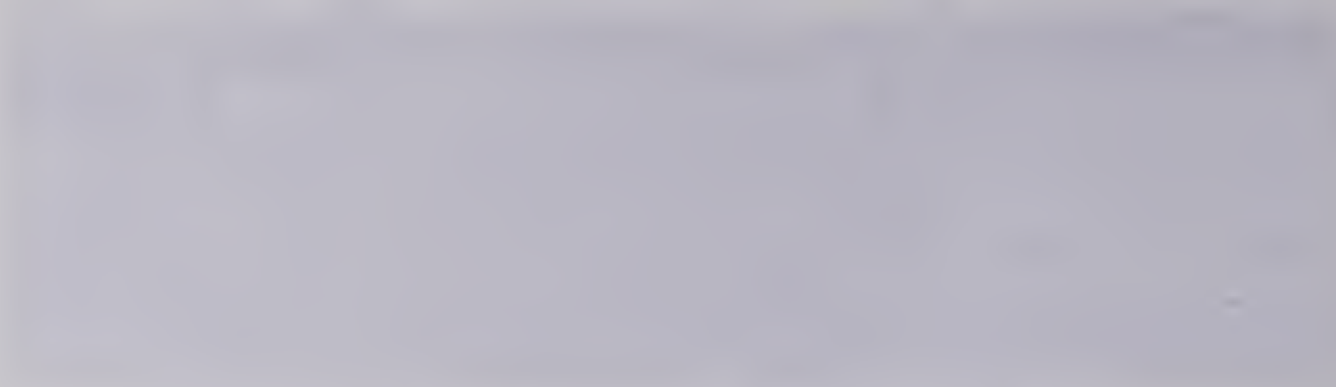
(a) Concrete - sack revetment
X- SEC 93
June 7, 1965 - Q≈ 25,000 cfs

(b) Water injection well being eroded
Revetment completely failed.
June 19, 1965 - 4:30 pm.
Q≈ 47,000 cfs.

(c) Erosion at X-SEC 93
July 7, 1965 - Q≈ 30,000 cfs

PHOTOGRAPH 7

EROSION OF REVETMENT
(North Sask. River)



(a) Q=7,000 cfs, MAY 19, 1965

600 0 600 ft.

(b) Q=7,000 cfs, SEPT. 17, 1965

PHOTOGRAPH 8

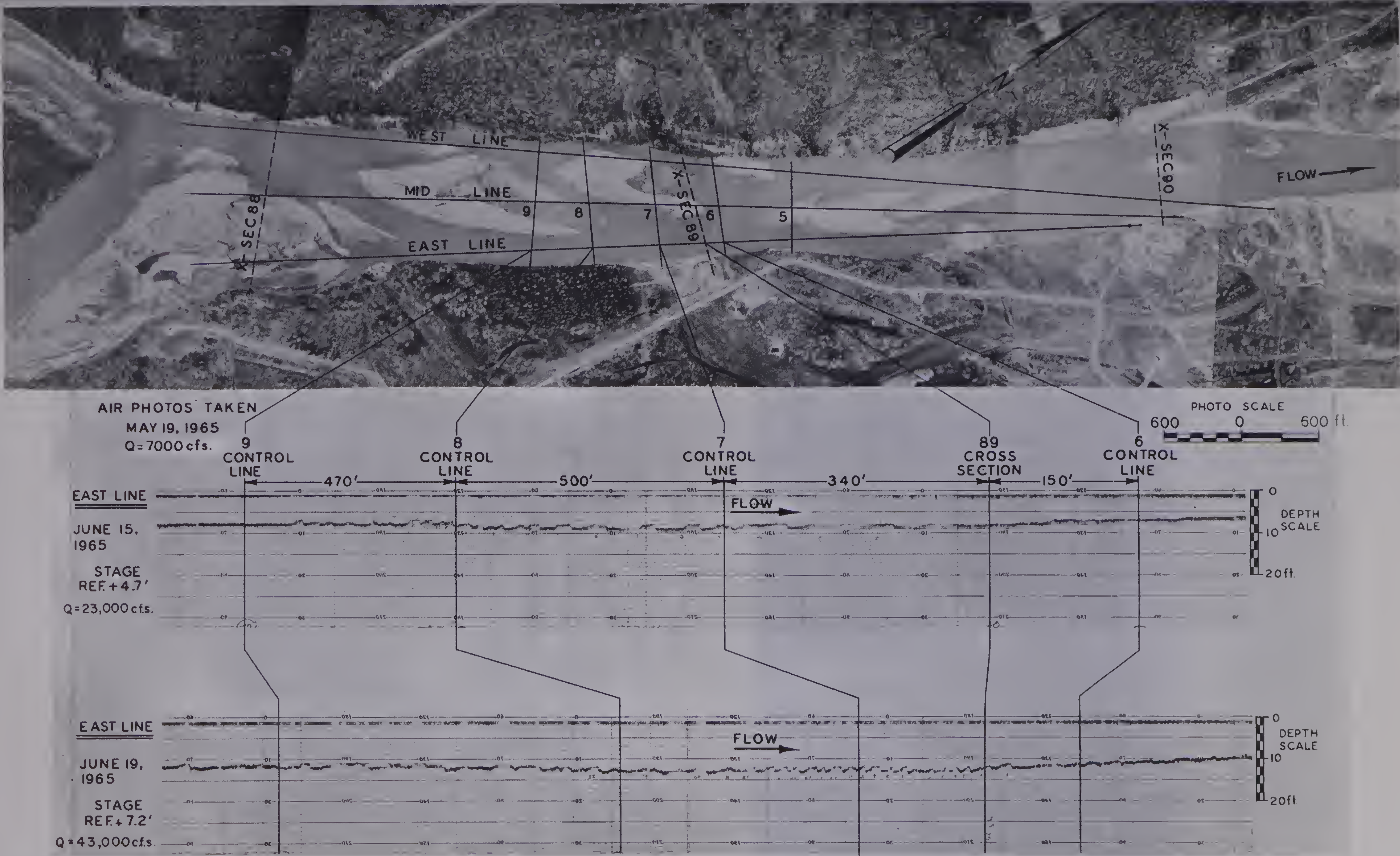
EROSION OF REVETMENT AND CHANNEL SHIFT
(North Sask. River)

PHOTOGRAPH 9

CONCRETE SACK ON TOP OF POINT BAR

SQUARE IS 2 FT.X 2 FT.

(North Sask. River, Drayton Valley)
x-sec 94



PHOTOGRAPH 10

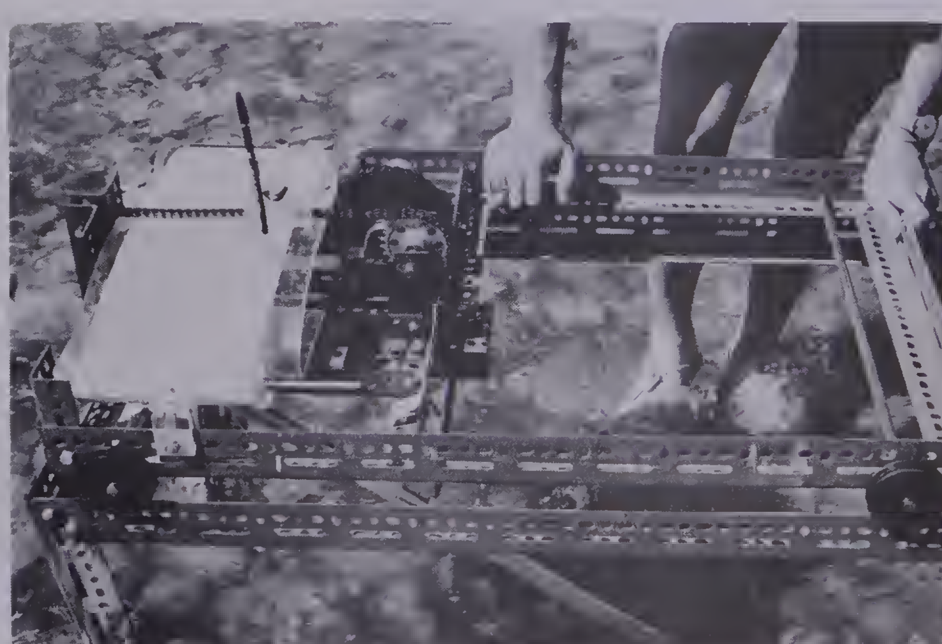
SOUNDINGS IN STUDY REACH
(North Sask. River)

PHOTOGRAPH 11

ARMOURED BED SHOWING SHINGLED PATTERN OF PARTICLES.THE BAR IS LOCATED IN THE MIDDLE OF THE RIVER AND IS PRESENTLY BEING ERODED.

(North Sask. River, Drayton Valley)
x-sec 92

(a) BED ROUGHNESS METER
THE METER ARM MOVES OVER STONES RECORDING THE PROJECTION HEIGHT OF THE STONE INTO THE FLOW.



(b) BED ROUGHNESS METER IN OPERATION

PHOTOGRAPH 12

(a) NATURAL SORTED GRAVEL BED

(b) ARTIFICIAL CEMENTED BED

PHOTOGRAPH 13

RESISTANCE TO FLOW- FLUME EXPERIMENTS

APPENDICES

APPENDIX 1

LIST OF SYMBOLS

SYMBOLS

A	Flow area (sq. ft.)
a	Major axis of bed-material particle
a	Constant
b	Channel width (ft)
b	Intermediate axis of bed-material particle
b_w	Water surface width (ft)
C	Bed load charge (pt. per 1000,000 by weight)
C_1	Constant
C_W	Concentration of suspended particles
c	Minor axis of bed-material particle
D	Particle size (ft) of bed-material
Di	The particle size corresponding to the ith percentile on the particle size distribution curve for bed-material (ft)
D_G	Geometric mean particle size
Dm	Representative particle size (ft)
d_*	Mean flow depth = $^A/b_w$ (ft)
d_b	Bankfull depth (ft)
ds	Depth of scour (ft)
Fb	Bed factor (Blench)
Fb_o	Zero bed factor (Blench)
Fs	Side Factor (Blench

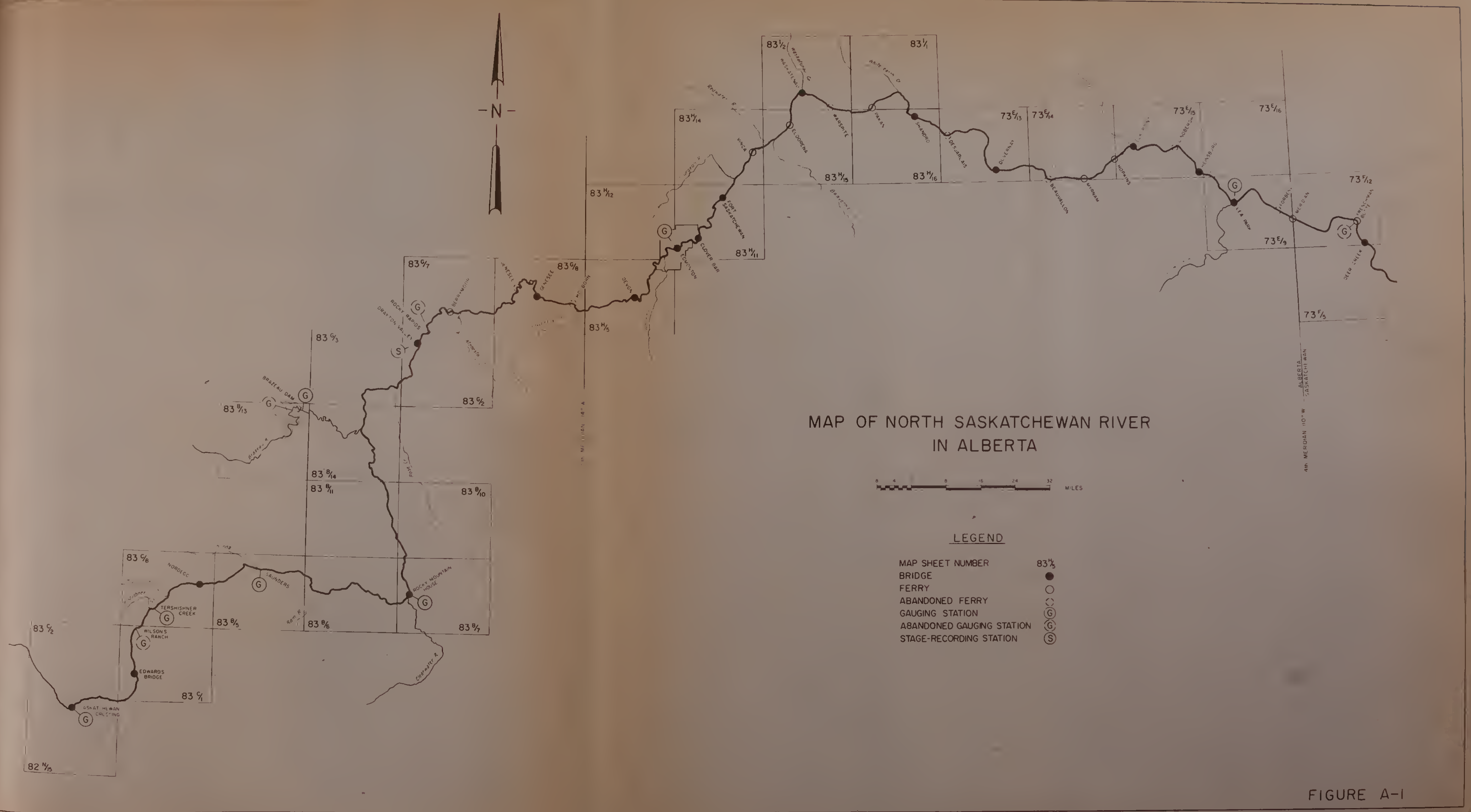
f	Function
f	Lacey's silt factor
fc	Factor defining cross-sectional shape of channel
fg	Factor defining plan geometry of channel
g	Acceleration of gravity (ft/sec^2)
k	von Karman turbulence coefficient
ks	Equivalent grain size (often D_{90})
k	Protrusion height of bed-material
M	Type of bank material parameter
n	Manning's roughness coefficient
P	Dimensionless bed-load transport parameter $(\frac{qs}{\rho v_*{}^3})$
P	Wetted Perimeter (ft)
Q	Discharge (ft^3/sec)
Qb	Bankfull discharge (ft^3/sec)
Qd	Dominant discharge (ft^3/sec)
qs	Weight of bed-load transported per unit time per unit width (lb/ft.sec)
R	Hydraulic radius (ft)
R_b	Hydraulic radius of the bed (ft)
R'_b	Hydraulic radius with respect to particle (ft.)
Rn	Reynolds number
r	Radius of curvature of river bend (ft)
S	Slope of energy grade line
Vm	Mean flow velocity based (ft/sec)
Vmc	Critical mean flow velocity for the threshold of motion of bed-material (ft/sec)

v_*	Shear velocity (ft/sec)
Wp	Weight of particle (lb)
w	Fall velocity of particle (ft/sec)
X	Particle Reynolds number $(\frac{\rho D v_*}{\mu})$
Y	Mobility number $(\frac{\rho v_*^2}{\gamma_s' D})$
Z	Relative depth $(\frac{d_*}{D})$
αb	Shape factor of bed-material
γ	Specific weight of water $(lb/_{ft}3)$
γ_s'	Bouyant weight of bed-material $(lb/_{ft}3)$
θ	Internal angle of curvature
λ	Concentration of bed-material or protrusions
μ	Dynamic viscosity of water sediment mixture (lb sec/ft^2)
ν	Kinematic viscosity of water sediment mixture (ft^2/sec)
ρ	Mass density of fluid (lb sec^2/ft^4)
ρs	Mass density of bed-material (lb sec^2/ft^4)
σb	Gradation of bed-material
σg	Geometric standard deviation
τo	Shear stress acting on the bed $(lb/_{ft}2)$
τc	Critical shear stress acting on the bed for the threshold of motion of bed-material $(lb/_{ft}2)$
ϕ	Einstein's bed-load function
χ	Areal pattern of bed-material or protrusions
ψ	Einstein's intensity of shear parameter

APPENDIX 2

NORTH SASKATCHEWAN RIVER
NEAR
DRAYTON VALLEY

GEOMORPHIC AND HYDRAULIC DATA



MAP OF NORTH SASKATCHEWAN RIVER
IN ALBERTA
LEGEND
MAP SHEET NUMBER
BRIDGE
FERRY
ABANDONED FERRY
GAUGING STATION
ABANDONED GAUGING STATION
STAGE-RECORDING STATION
MILES
N
FORT SASKATCHEWAN
CLOVER BAR
EDMONTON
DEVON
ELDORENA
VINCA
SHANDRO
DESJARLAIS
DUVERNAY
BEAUVALLON
MYRNAM
HOPKINS
LEA PARK
ROCKY RAPIDS
DRAYTON VALLEY
BRAZEAU DAM
ROCKY MOUNTAIN HOUSE
SAUNDERS
NORDEGG
TERSHISHNER CREEK
WILSONS RANCH
EDWARDS BRIDGE
ALBERTA
SASKATCHEWAN
4th MERIDIAN 110° W
Clearwater R.

FIGURE A-1

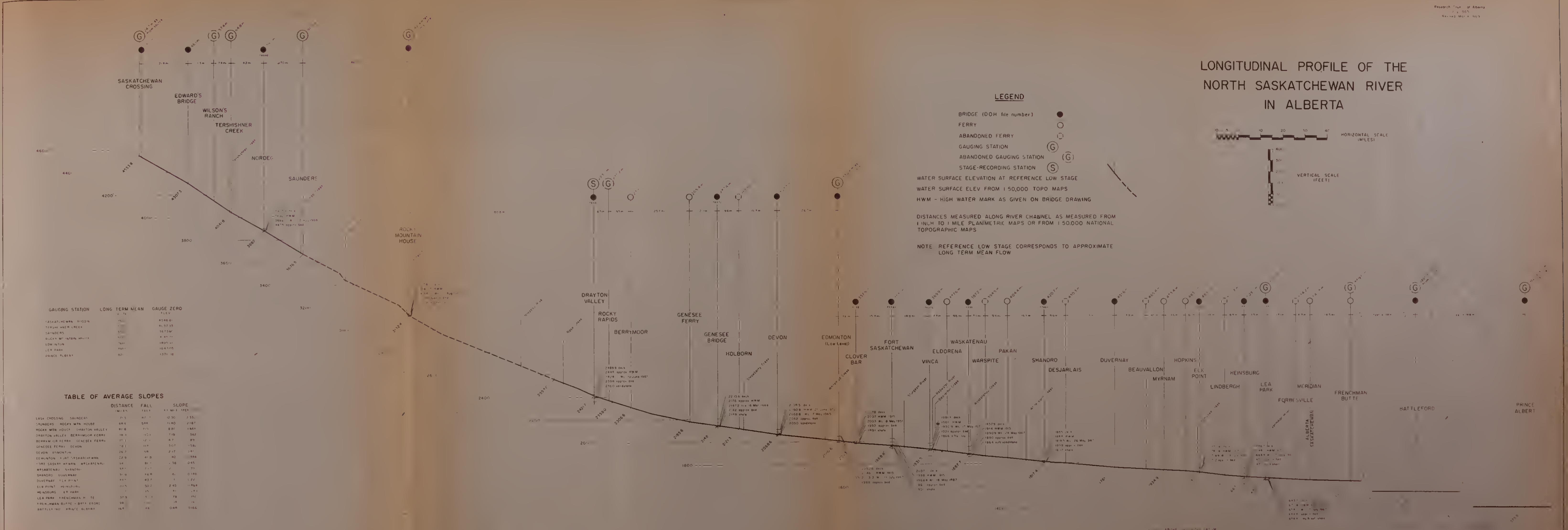

LONGITUDINAL PROFILE OF THE
NORTH SASKATCHEWAN RIVER
IN ALBERTA
HORIZONTAL SCALE (MILES)
VERTICAL SCALE (FEET)
LEGEND
BRIDGE (DOH file number)
FERRY
ABANDONED FERRY
GAUGING STATION
ABANDONED GAUGING STATION
STAGE-RECORDING STATION
WATER SURFACE ELEVATION AT REFERENCE LOW STAGE
WATER SURFACE ELEV FROM 1:50,000 TOPO MAPS
HWM - HIGH WATER MARK AS GIVEN ON BRIDGE DRAWING
DISTANCES MEASURED ALONG RIVER CHANNEL AS MEASURED FROM 1 INCH TO 1 MILE PLANIMETRIC MAPS OR FROM 1:50,000 NATIONAL TOPOGRAPHIC MAPS
NOTE: REFERENCE LOW STAGE CORRESPONDS TO APPROXIMATE LONG TERM MEAN FLOW
SASKATCHEWAN CROSSING
EDWARD'S BRIDGE
WILSON'S RANCH
TERSHISHNER CREEK
NORDEG
SAUNDERS
ROCKY MOUNTAIN HOUSE
DRAYTON VALLEY
ROCKY RAPIDS
BERRYMOOR
GENESEE FERRY
GENESEE BRIDGE
HOLBORN
DEVON
EDMONTON (Low Level)
CLOVER BAR
FORT SASKATCHEWAN
VINCA
ELDORENA
WASKATENAU
WARSPITE
PAKAN
SHANDRO
DESJARLAIS
DUVERNAY
BEAUVALLON
MYRNAM
HOPKINS
ELK POINT
LINDBERGH
HEINSBURG
LEA PARK
FORT SASKATCHEWAN
MERIDIAN
FORRESVILLE
ALBERTA
SASKATCHEWAN
FRENCHMAN BUTTE
BATTLEFORD
PRINCE ALBERT
GAUGING STATION
LONG TERM MEAN
GAUGE ZERO
TABLE OF AVERAGE SLOPES
DISTANCE
FALL
SLOPE
SASK CROSSING - SAUNDERS
SAUNDERS - ROCKY MTN HOUSE
ROCKY MTN HOUSE - DRAYTON VALLEY
DRAYTON VALLEY - BERRYMOOR FERRY
GENESEE FERRY - DEVON
DEVON - EDMONTON
SHANDRO - DUVERNAY
HEINSBURG
LEA PARK
4600
4400
4200
4000
3800
3600
3400
3200
2400
1800
1600

FIGURE A-2

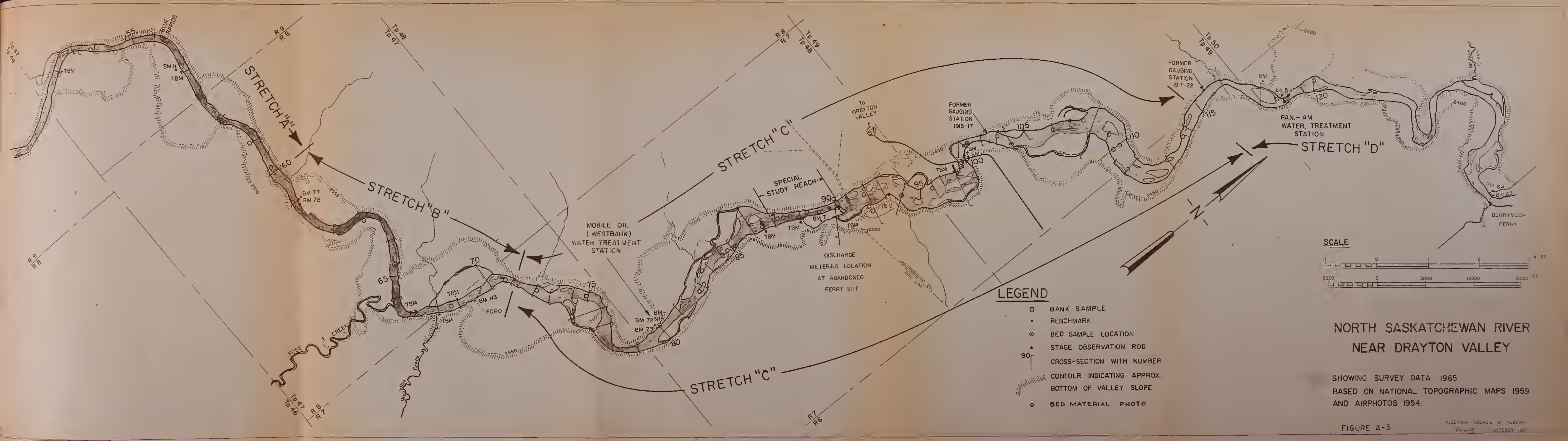
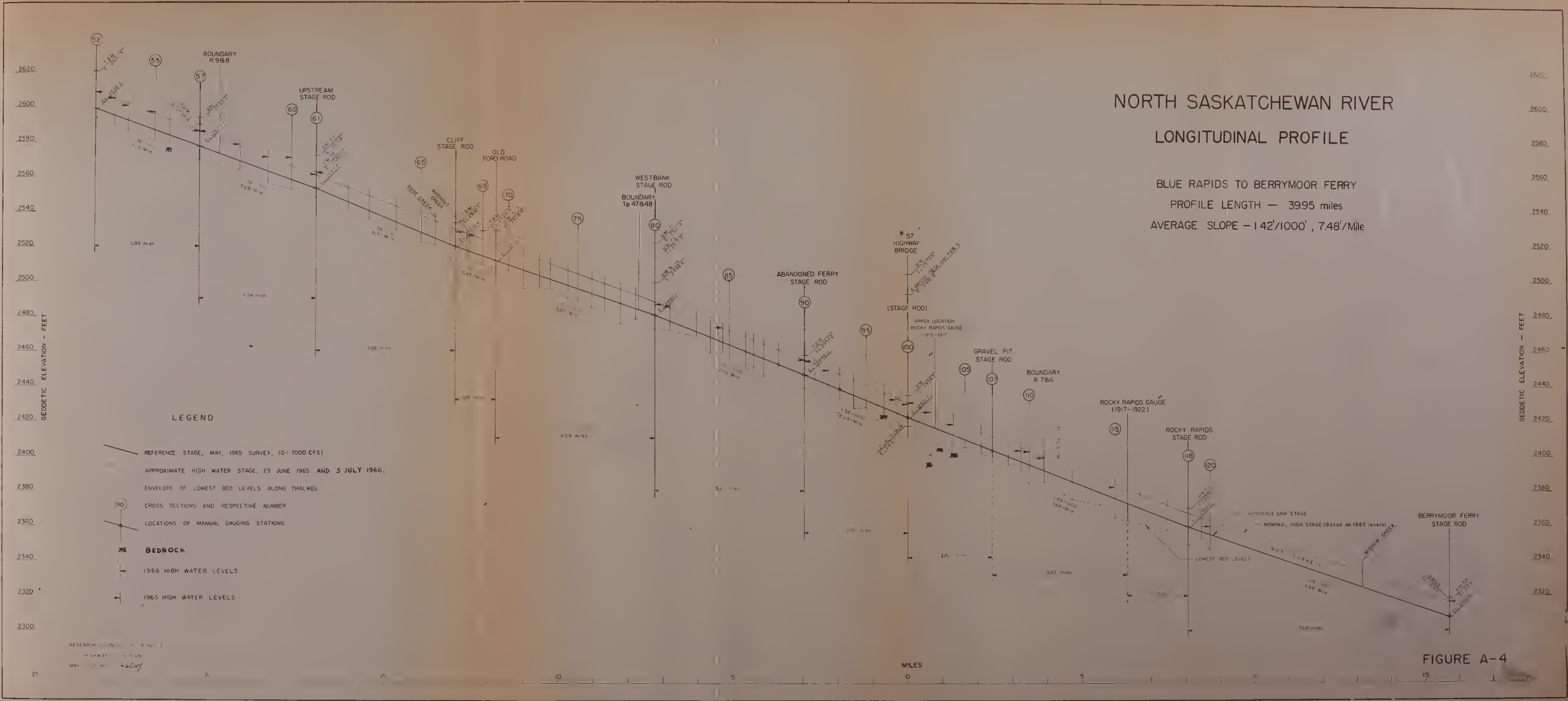
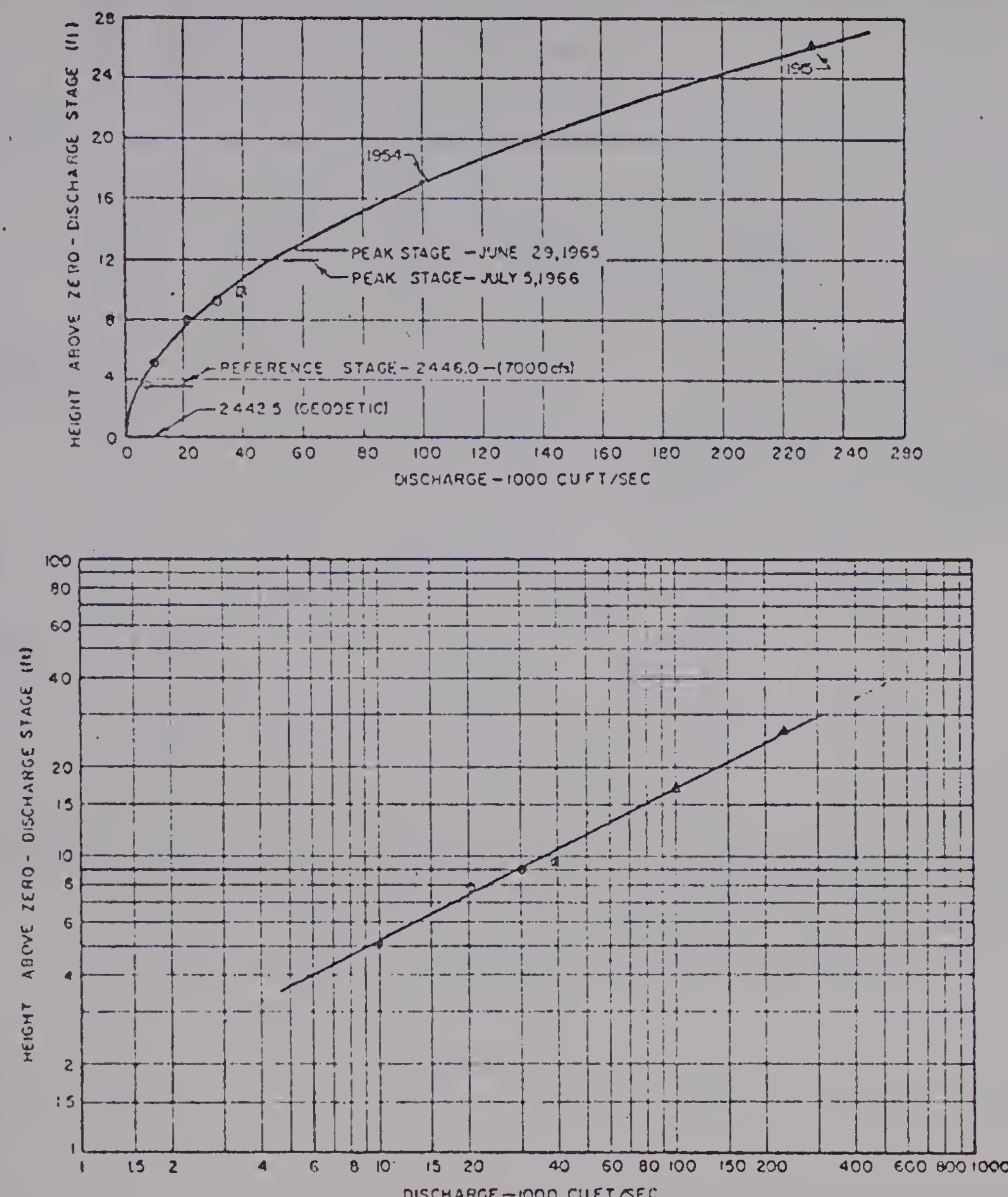

NORTH SASKATCHEWAN RIVER
NEAR DRAYTON VALLEY
SHOWING SURVEY DATA 1965
BASED ON NATIONAL TOPOGRAPHIC MAPS 1959
AND AIRPHOTOS 1954.
LEGEND
BANK SAMPLE
BENCHMARK
BED SAMPLE LOCATION
STAGE OBSERVATION ROD
CROSS-SECTION WITH NUMBER
CONTOUR INDICATING APPROX. BOTTOM OF VALLEY SLOPE
BED MATERIAL PHOTO
SCALE
STRETCH "A"
STRETCH "B"
STRETCH "C"
STRETCH "D"
SPECIAL STUDY REACH
MOBILE OIL (WESTBANK) WATER TREATMENT STATION
PAN - AM WATER TREATMENT STATION
FORMER GAUGING STATION 1915-17
FORMER GAUGING STATION 1917-22
DISCHARGE METERING LOCATION AT ABANDONED FERRY SITE
To DRAYTON VALLEY
BLUE RAPIDS
BERRYMOOR FERRY
ROSE CREEK
UNDERGROUND OIL PIPE LINE
RESEARCH COUNCIL OF ALBERTA

FIGURE A-3



NORTH SASKATCHEWAN RIVER
LONGITUDINAL PROFILE
BLUE RAPIDS TO BERRYMOOR FERRY
PROFILE LENGTH — 39.95 miles
AVERAGE SLOPE – 1.42'/1000', 7.48'/Mile
GEODETIC ELEVATION – FEET
MILES
BOUNDARY R 988
UPSTREAM STAGE ROD
CLIFF STAGE ROD
OLD FORD ROAD
ROSE CREEK
WASHOUT CREEK
WESTBANK STAGE ROD
BOUNDARY Tp 47848
ABANDONED FERRY STAGE ROD
57 HIGHWAY BRIDGE
(STAGE ROD)
APPROX. LOCATION ROCKY RAPIDS GAUGE 1915–1917
GRAVEL PIT STAGE ROD
BOUNDARY R 786
ROCKY RAPIDS GAUGE (1917–1922)
ROCKY RAPIDS STAGE ROD
REFERENCE LOW STAGE
NOMINAL HIGH STAGE (Based on 1965 levels)
LOWEST BED LEVELS
NOT SURVEYED
BERRYMOOR FERRY STAGE ROD
LEGEND
REFERENCE STAGE, MAY, 1965 SURVEY, (0-7000 CFS)
APPROXIMATE HIGH WATER STAGE, 29 JUNE 1965 AND 5 JULY 1966.
ENVELOPE OF LOWEST BED LEVELS ALONG THALWEG
CROSS SECTIONS AND RESPECTIVE NUMBER
LOCATIONS OF MANUAL GAUGING STATIONS
BEDROCK
1966 HIGH WATER LEVELS
1965 HIGH WATER LEVELS

FIGURE A-4



NORTH SASKATCHEWAN RIVER NEAR DRAYTON VALLEY

STAGE - DISCHARGE CURVE (TENTATIVE)

ABANDONED FERRY SITE
X-SEC 90, MILE 221

LEGEND

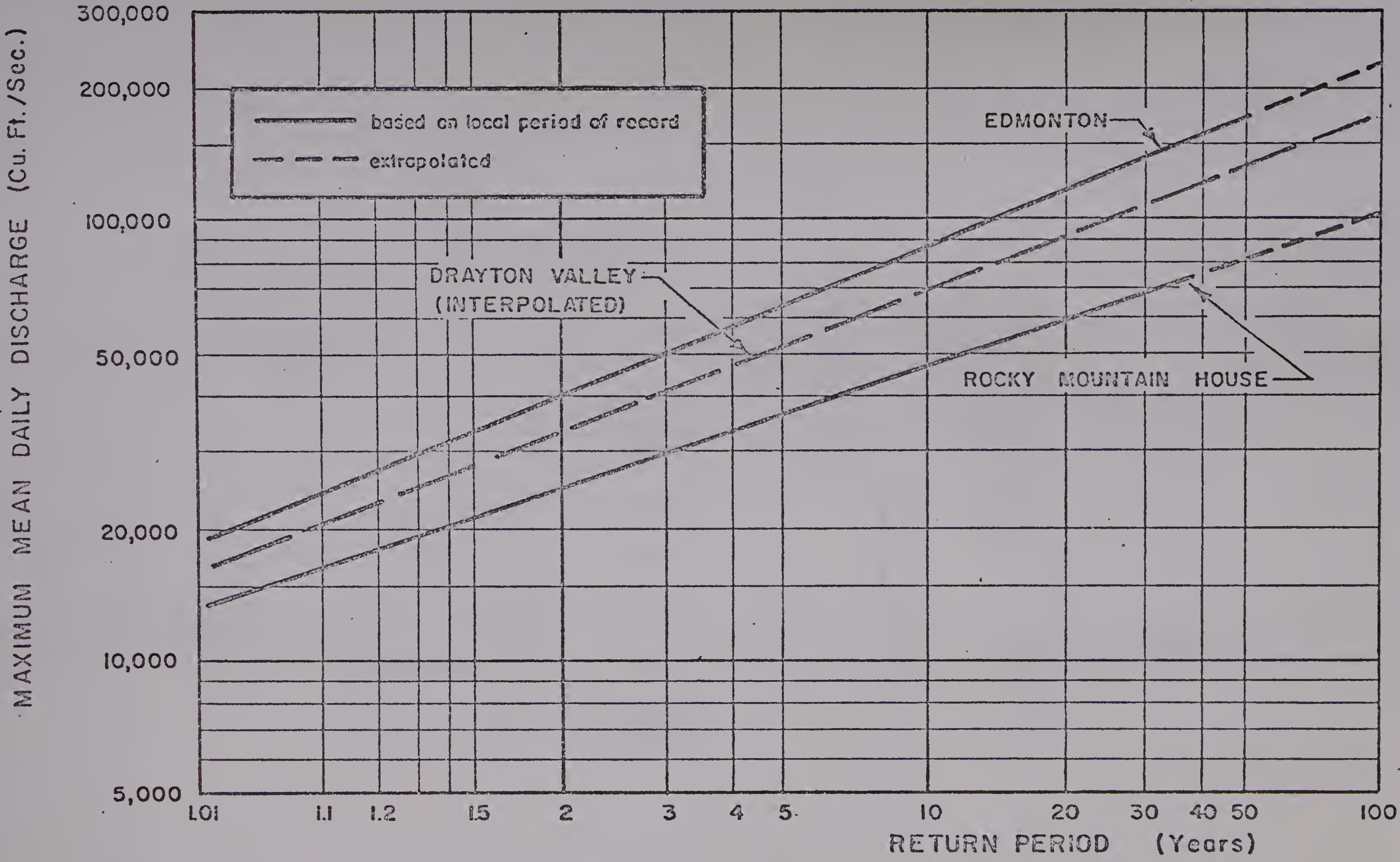
- o MEASURED DISCHARGES – 1965
- □ MEASURED DISCHARGE – 1966
- △ ESTIMATED HISTORICAL FLOWS

NOTES

1. REFERENCE STAGE ESTABLISHED AT 7,000cfs.
2. ALL ELEVATIONS GEODETIC.
3. DISCHARGE MEASURED:

   JUNE 4, 1965 – 30,900 cfs. – 2451.0' elev.
   AUG. 4, 1965 – 20,550 cfs. – 2449.5'
   AUG. 20, 1965 – 10,050 cfs. – 2447.7'
   JULY 7, 1966 – 39,820 cfs. – 2451.9'

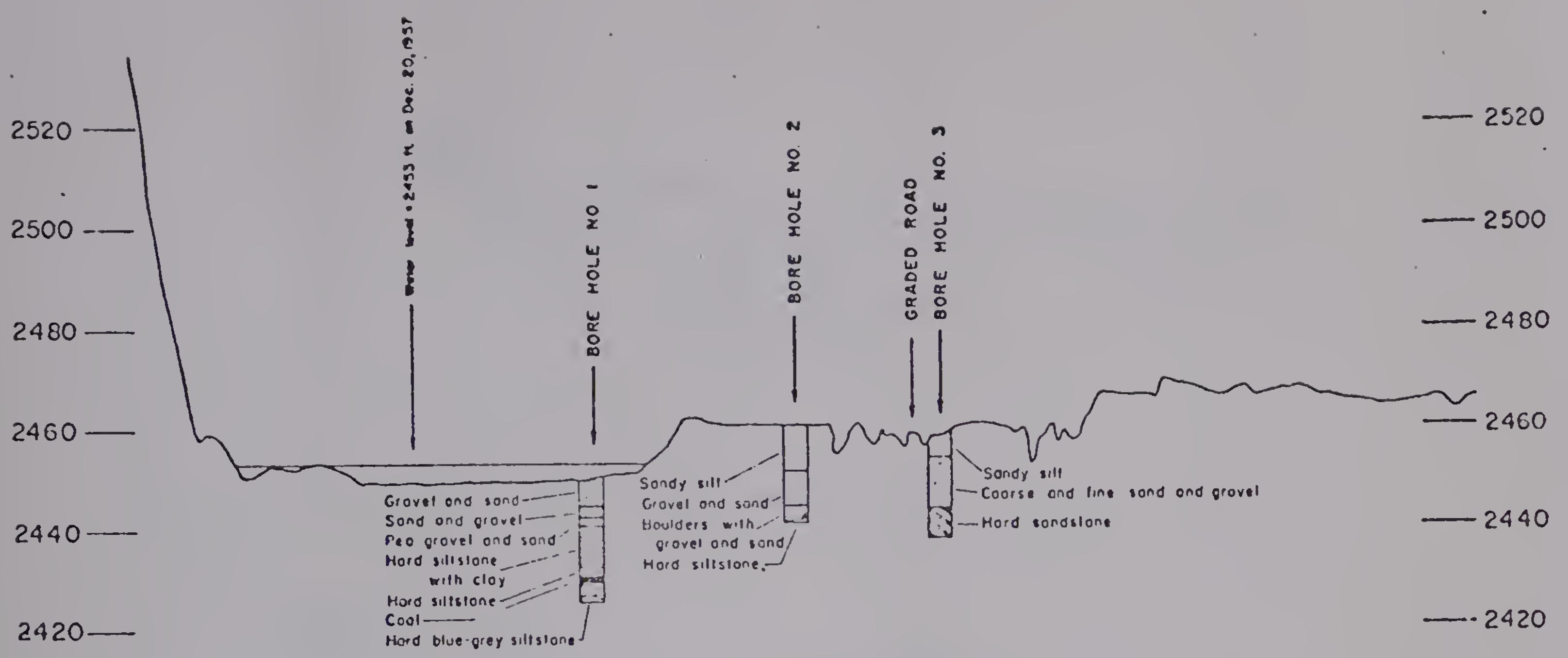
FIGURE A-5



NORTH SASKATCHEWAN RIVER
FREQUENCY CURVES

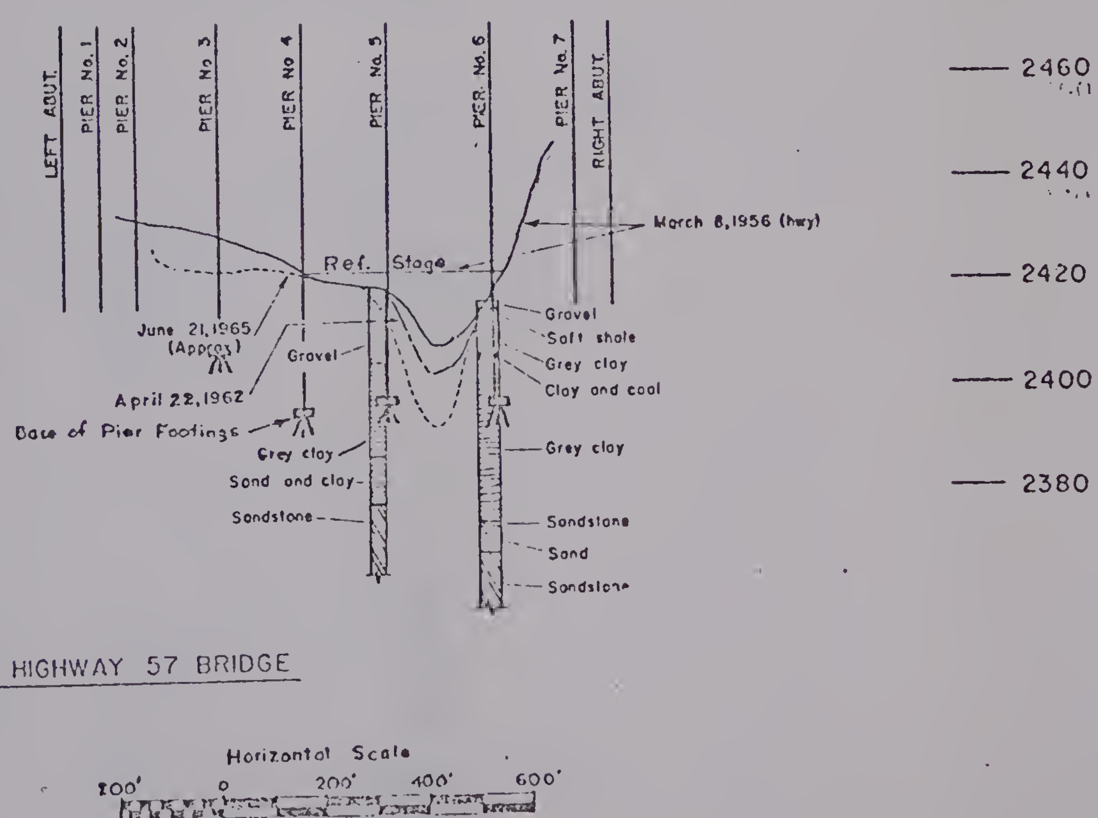
(LOG. GUMBEL PLOTS)

FIGURE A-6



Cross-section by Stewert, Weir, and Stewert (Surveyors), Dec. 1957
Borehole data by Hardy and Assoc, 4 Dec 1957

NEAR X-SECTION NO.89

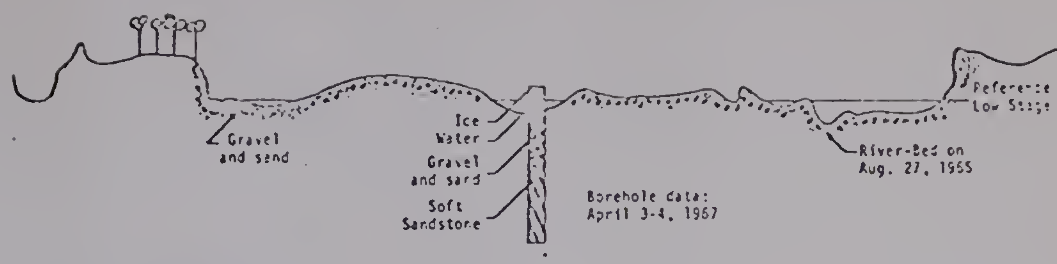


Borehole data by Alta Dept of Highways, 8 March 1956

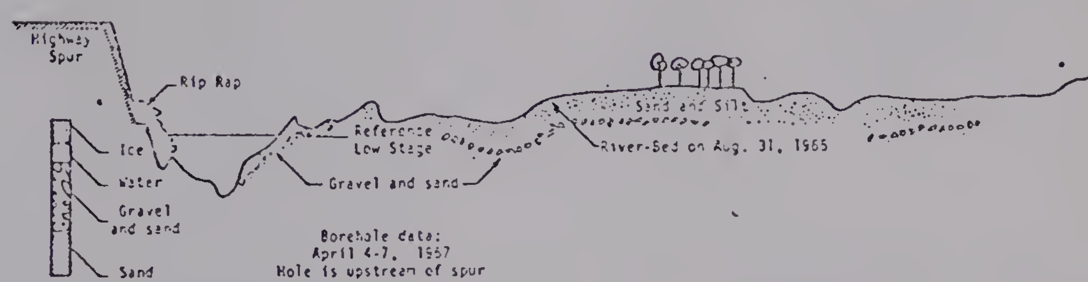
AT HIGHWAY 57 BRIDGE

BOREHOLE DATA (Sections viewed downstream)

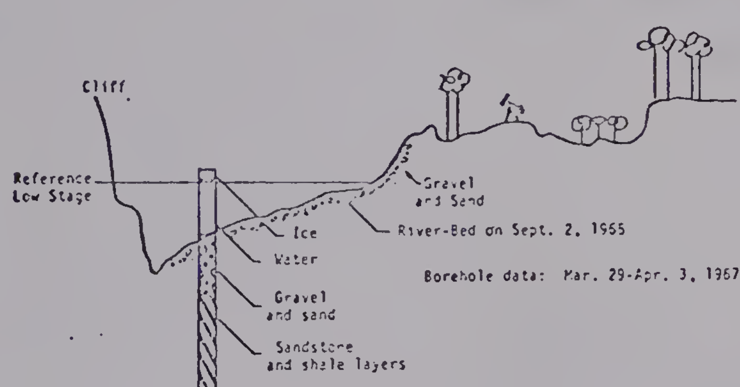
FIGURE A-7



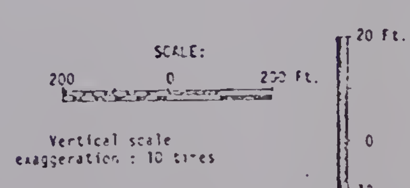
AT X-SEC NO. 93



AT X-SEC NO. 98



AT X- SEC NO. 102



DATA SUPPLIED BY BRIDGE BRANCH, ALBERTA DEPARTMENT OF HIGHWAYS

BOREHOLE DATA

Sections viewed downstream

FIGURE A-8

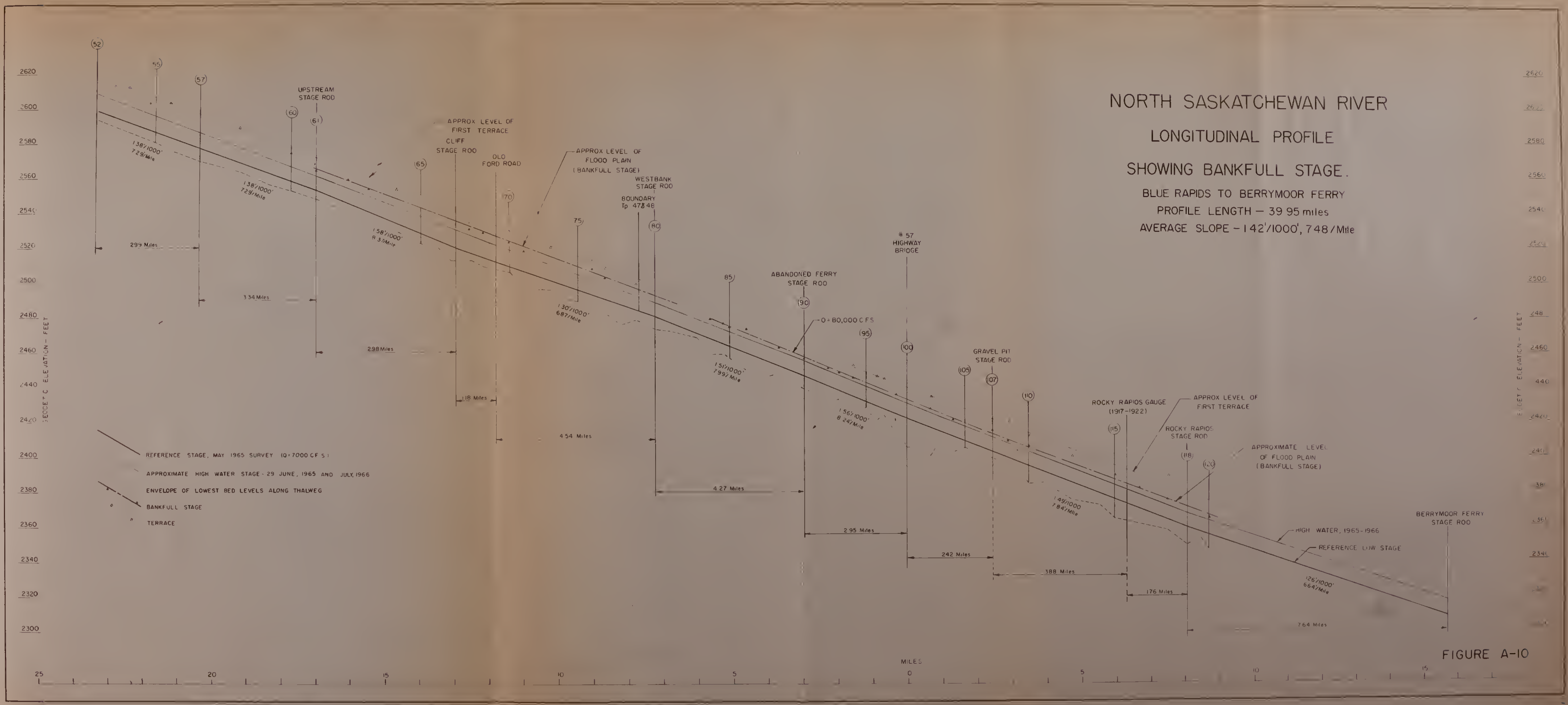

NORTH SASKATCHEWAN RIVER
LONGITUDINAL PROFILE
SHOWING BANKFULL STAGE.
BLUE RAPIDS TO BERRYMOOR FERRY
PROFILE LENGTH – 39.95 miles
AVERAGE SLOPE – 1.42'/1000', 7.48/Mile
GEODETIC ELEVATION – FEET
MILES
UPSTREAM STAGE ROD
APPROX LEVEL OF FIRST TERRACE
CLIFF STAGE ROD
OLD FORD ROAD
APPROX LEVEL OF FLOOD PLAIN (BANKFULL STAGE)
WESTBANK STAGE ROD
BOUNDARY Tp 47 & 48
ABANDONED FERRY STAGE ROD
Q = 80,000 C.F.S.
# 57 HIGHWAY BRIDGE
GRAVEL PIT STAGE ROD
ROCKY RAPIDS GAUGE (1917–1922)
APPROX LEVEL OF FIRST TERRACE
ROCKY RAPIDS STAGE ROD
APPROXIMATE LEVEL OF FLOOD PLAIN (BANKFULL STAGE)
HIGH WATER, 1965–1966
REFERENCE LOW STAGE
BERRYMOOR FERRY STAGE ROD
1.38'/1000' 7.29'/Mile
1.38'/1000' 7.29'/Mile
1.58'/1000' 8.37/Mile
1.30'/1000' 6.87/Mile
1.51'/1000' 7.99'/Mile
1.56'/1000' 8.24/Mile
1.49'/1000' 7.84'/Mile
1.26'/1000' 6.64'/Mile
2.99 Miles
3.34 Miles
2.98 Miles
1.18 Miles
4.54 Miles
4.27 Miles
2.95 Miles
2.42 Miles
3.88 Miles
1.76 Miles
7.64 Miles
REFERENCE STAGE, MAY 1965 SURVEY (Q=7000 C.F.S.)
APPROXIMATE HIGH WATER STAGE - 29 JUNE, 1965 AND JULY, 1966
ENVELOPE OF LOWEST BED LEVELS ALONG THALWEG
BANKFULL STAGE
TERRACE
FIGURE A-10

TABLE A-1

NORTH SASKATCHEWAN RIVER
NEAR DRAYTON VALLEY
CHANNEL CROSS SECTION DIMENSIONS

River Stretch	Cross Section No.	Height Ref. to Peak-Flow Stage (ft.)	REFERENCE STAGE			PEAK-FLOW STAGE (1965-66)			
			Cross Section Area (ft.^{2})	Water-Surface Width (ft.)	Mean Depth (ft.) A/W_s	Cross Section Area (ft.^{2})	Water-Surface Width (ft.)	Mean Depth (ft.) A/W_s	Maximum Depth (ft.)
A	52	9.5	960	570	1.5	6800	638	10.7	14.0
	53	9.5	2160	513	4.2	7900	624	12.7	15.0
	54	9.5	1700	515	3.3	8020	775	10.4	15.7
	55	9.0	2200	500	4.4	7240	590	12.3	15.0
	56	9.0	2120	407	5.2	7100	710	10.0	16.5
	57	9.0	2080	520	4.0	7180	600	12.0	16.2
	58	8.5	1200	325	3.7	5840	820	7.1	13.3
	59	8.5	1840	377	4.9	6030	670	9.1	15.0
	60	8.0	1500	383	3.9	6760	845	8.0	13.5
B	61	7.5	920	280	3.3	8120	1020	8.0	13.2
	62	8.5	1320	640	2.1	7100	830	8.6	12.5
	63	8.5	1340	450	3.0	7120	710	10.0	12.7
	64	9.0	580	190	3.1	7860	840	9.4	13.6
	65	9.3	1280	440	2.9	7420	740	10.0	15.6
	66	10.0	1000	590	1.7	8320	855	9.7	13.0
	67	10.0	1400	325	4.3	5720	520	11.0	18.0
	68	9.5	1100	560	2.0	8600	880	9.8	14.0
	69	9.3	1440	380	3.8	8440	920	9.2	14.8
	70	9.0	700	370	1.9	7520	930	9.1	12.2
	71	9.0	1520	250	6.1	4460	450	9.3	19.3
C	72	9.0	2100	450	4.7	12280	1390	8.8	15.8
	73	9.0	2140	530	4.0	12140	1214	10.0	13.9
	74	8.5	2420	620	3.9	13500	1315	10.3	16.2
	75	8.5	2000	440	4.6	6660	640	10.4	14.8
	76	8.3	2780	450	6.2	6600	510	12.9	16.5
	77	8.0	2140	375	5.7	5900	560	10.5	16.3
	78	8.0	2200	420	5.3	6200	553	11.2	19.1
	79	8.0	1840	500	3.7	7660	725	11.6	14.2
	80	7.5	1900	490	3.9	6620	685	9.7	15.0
	81	7.5	1340	430	3.1	6760	720	9.4	14.1
	82	8.0	440	225	1.5	6100	1170	5.2	10.1
	83	8.0	1722	290	5.9	5142	650	7.9	17.0
	84	8.0	1480	590	2.5	9100	845	10.8	13.4
	85	8.0	1960	400	4.9	7620	850	9.0	14.0
	86	8.0	1840	360	5.1	6340	720	8.9	17.9
	87	8.0	1440	503	2.9	5700	545	10.5	16.0
	88	8.0	2060	330	6.2	6800	680	10.0	19.0
	89	8.3	1200	365	3.3	7460	810	9.2	13.0
	90	8.5	2460	650	3.8	8100	745	10.9	15.8
	91	8.5	1940	475	4.1	5160	725	7.1	16.2
	92	8.5	2200	685	3.2	8520	760	11.2	14.8
	93	8.5	1000	460	2.2	11480	1440	8.0	13.3
	94	8.5	1860	430	4.3	9400	1310	7.2	15.9
	95	8.0	1600	460	3.5	7300	740	9.9	13.0
	96	8.0	1940	490	4.0	8920	890	10.0	17.3
	97	8.0	1220	360	3.4	9380	1040	9.0	12.5
	98	8.3	1360	225	6.0	6220	780	8.0	20.3
	99	8.3	2650	415	6.4	9140	960	9.5	17.5
	100	8.0	2540	290	8.8	7280	715	10.2	30.5
	101	8.0	850	470	1.8	9580	1013	9.5	21.5
	102	8.0	3700	510	7.2	7720	570	13.5	24.8
	103	8.0	2240	420	5.3	10420	1055	9.9	18.0
	104	8.0	2120	460	4.6	12860	1840	7.0	15.3
	105	8.0	950	416	2.3	8990	1182	7.6	12.0
	106	7.8	1580	540	2.9	7220	770	9.4	12.2
	107	7.8	760	240	3.2	5200	782	6.7	12.5
	108	7.8	1480	600	2.5	6960	740	9.4	12.5
	109	7.5	1400	530	2.6	7040	810	8.7	13.5
	110	7.5	1940	400	4.9	8140	1125	7.2	18.0
	111	7.5	1680	350	4.8	10480	1338	7.8	14.5
	112	-	-	-	-	-	-	-	-
	113	8.0	1120	330	3.4	10080	1000	10.1	16.5
	114	7.8	2050	515	4.0	8050	880	9.2	14.0
	115	7.8	3080	410	7.5	10060	1030	9.8	19.0
D	116	7.8	2000	370	5.4	7780	845	9.2	16.2
	117	7.5	2050	600	3.4	7020	730	9.6	14.0
	118	8.0	1880	490	3.8	5820	550	10.6	17.5
	119	7.8	1480	550	2.7	9430	1130	8.4	11.8
	120	7.8	2640	505	5.2	9440	1070	8.8	15.0

TABLE A-2

AVERAGE CHANNEL DIMENSIONS
FOR FOUR RIVER STRETCHES

RIVER STRETCH	NO. OF CROSS SECTIONS	HEIGHT REF. TO NOM. HIGH STAGE	REFERENCE LOW STAGE					NOMINAL HIGH STAGE				
			AREA - (A)		WIDTH - (Ws)		DEPTH A/Ws	AREA - (A)		WIDTH - (Ws)		DEPTH = A/Ws
		ft.	AVERAGE sq. ft.	COEFF. OF VARIATION	AVERAGE sq. ft.	COEFF. OF VARIATION	ft.	AVERAGE sq. ft.	COEFF. OF VARIATION	AVERAGE ft.	COEFF. OF VARIATION	ft.
A	9	9.0	1,750	$\pm$26%	460	$\pm$18%	3.8	6,990	$\pm$11%	700	$\pm$14%	10.0
B	11	9.0	1,145	$\pm$27%	410	$\pm$12%	2.8	7,335	$\pm$17%	795	$\pm$21%	9.2
C	43	8.0	1,832	$\pm$35%	440	$\pm$24%	4.2	8,195	$\pm$26%	905	$\pm$29%	9.1
D	5	7.8	2,010	$\pm$21%	500	$\pm$17%	4.0	7,910	$\pm$20%	865	$\pm$28%	9.1

TABLE A-3

NORTH SASKATCHEWAN RIVER

NEAR DRAYTON VALLEY

ANALYSES OF COARSE BED-MATERIAL

LOCATION		ANALYSES				SAMPLING TECHNIQUE
Cross Section	Part of Channel	Median Size (in)	Type of Analysis	Mean Size (in)	Sorting (Geom. Standard dev.)	
85 D/S	Right point bar	2.2	% by number	2.3	1.50	Line (tape) - 2 ft.
87 D/S	Right point bar	2.3	% by number	2.5	1.78	Line (tape) - 2 ft.
"	"	2.4	% by number	2.6	1.67	Line (pace) - 3 ft.
87 D/S	In channel	2.4	% by number	2.6	1.71	Line (pace) - 3 ft.
90 U/S	Right point bar	1.4	% by number	1.5	1.71	Line (tape) - 1 ft.
"	"	1.1	% by weight	-	-	Sub-surface scoop
90	Right point bar	1.1	% by number	1.2	1.81	Line (tape) - 1 ft.
"	"	0.7	% by weight	-	-	Sub-surface scoop
94 U/S	Right point bar	1.2	% by number	1.3	1.66	Grid (lines)
"	"	0.8	% by weight	-	-	Sub-surface scoop
94	"	1.6	% by number	1.8	1.87	Line (tape) - 1 ft.
"	"	0.9	% by weight	-	-	Sub-surface scoop
100	Left point bar	2.0	% by number	2.1	1.50	Line (pace) - 3 ft.
"	"	1.7	% by number	1.9	1.60	Line (tape) - 2 ft.
100 U/S	Left Point bar	1.1	% by number	1.2	1.55	Line (tape) - 1 ft.
"	"	1.4	% by weight	-	-	Sub-surface scoop
100	Left point bar	1.9	% by number	2.2	1.85	Line (tape) - 1 ft.
"	"	1.1	% by weight	-	-	Sub-surface scoop
100 D/S	Left point bar	1.4	% by number	1.5	1.70	Line (tape) - 1 ft.
"	"	1.0	% by weight	-	-	Sub-surface scoop
101	Right point bar	2.1	% by number	2.2	1.60	Line (tape) - 1 ft.
"	"	1.3	% by weight	-	-	Sub-surface scoop
101 D/S	Right point bar	1.9	% by number	-	-	Line (tape) - 1 ft.
"	"	1.5	% by weight	-	-	Sub-surface scoop
104	Right bar	1.5	% by number	1.6	1.85	Line (tape) - 1 ft.
"	"	1.1	% by weight	-	-	Sub-surface scoop
104 D/S	Right bar	1.1	% by number	1.2	1.55	Line (tape) - 1 ft.
"	"	0.7	% by weight	-	-	Sub-surface scoop

TABLE A-4

ANALYSES OF BANK-MATERIAL SAMPLES

LOCATION		ANALYSES				REMARKS
Cross Section	Part of Bank	Percentages of Gravel	Sand	Silt	Clay	
59	Upper part of right point bar	-	80	15	5	Recent deposition
92	Right eroding bank	1	44	-	55 -	Flood plain deposit, one foot below ground
92	Right flood plain	-	80	14	6	Four feet below ground
55	Left bank	-	77	-	23 -	Four feet above water, recent deposit

APPENDIX 3

RIVER DATA

THRESHOLD OF MOTION OF BED-MATERIAL

TABLE A-5

RIVER DATA - THRESHOLD OF MOTION OF BED-MATERIAL

RIVER	LOCATION	REFERENCE & PLOTTING SYMBOL	MAXIMUM DISCHARGE Q (cfs)	MEAN CRITICAL VELOCITY Vm_c (ft/sec)	PARTICLE SIZE D (ft)	SLOPE	MEAN DEPTH OF FLOW d_* (ft)	CRITICAL SHEAR STRESS τo_c (lb/ft²)	d_*/D	D/d_*	$\frac{\rho Vm_c^2}{\gamma_s D}$	SHEAR VELOCITY v_* (ft/sec)	$\frac{\rho v_*^2}{\gamma_s' D}$	$\frac{D v_*}{\nu} \times 10^4$	$Fbo = \frac{Vm_c^2}{d_*}$	COMMENTS
North Saskatchewan (Drayton Valley)		Galay (1967a)														
	# 94	○	60,000	6.4	0.8	0.0015	7.2	0.67	9.0	0.111	0.95	0.592	0.0081	3.95	5.70	Concrete Sack (S.G.= 2.4).
	# 81		60,000	9.0	0.7	0.0015	9.4	0.88	10.4	0.080	2.14	0.674	0.012	3.93	8.62	Boulders found on bed or on bars.
	# 85		60,000	7.9	0.8	0.0015	9.0	0.85	11.2	0.080	1.44	0.659	0.010	4.40	6.93	
	# 86		60,000	9.5	0.7	0.0015	8.9	0.83	11.4	0.079	2.38	0.655	0.011	3.82	10.14	D = average of largest material.
	# 88		60,000	10.5	0.6	0.0015	10.0	0.94	16.6	0.060	3.40	0.695	0.015	3.47	11.00	
	# 94		60,000	6.4	0.7	0.0015	7.2	0.67	10.3	0.097	1.08	0.590	0.009	3.44	5.70	
	# 96		60,000	6.7	0.9	0.0015	10.0	0.94	11.1	0.090	0.92	0.695	0.010	5.20	4.49	
	# 97		60,000	6.4	0.8	0.0015	9.0	0.85	11.2	0.089	0.95	0.659	0.010	4.40	4.55	
	#101		60,000	6.3	0.9	0.0015	9.5	0.39	10.5	0.095	0.82	0.678	0.010	5.08	4.29	
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Wilson Creek (Manitoba)		Newbury (1968)														
	Site 1	△	-	12.2	2.1	0.035	2.7	5.90	1.28	0.780	1.31	1.746	0.028	30.6	55.2	D = average of largest boulders, smaller than flow depth.
	Site 4		-	10.1	2.5	0.035	3.2	7.00	1.28	0.780	0.75	1.880	0.027	39.2	31.9	
	Site 6		-	9.1	1.6	1.0055	2.1	0.72	1.32	0.760	0.96	0.610	0.004	8.14	39.4	
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...

TABLE A-5

RIVER DATA - THRESHOLD OF MOTION OF BED-MATERIAL

RIVER	LOCATION	REFERENCE & PLOTTING SYMBOL	MAXIMUM DISCHARGE Q (cfs)	MEAN CRITICAL VELOCITY V_{m_c} (ft/sec)	PARTICLE SIZE D	SLOPE	MEAN DEPTH OF FLOW d_* (ft)	CRITICAL SHEAR STRESS τ_{o_c} (lb/ft²)	d_*/D	D/d_*	$\frac{\rho V_{m_c}^2}{\gamma_s' D}$	SHEAR VELOCITY V_* (ft/sec)	$\frac{\rho V_*^2}{\gamma_s' D}$	$\frac{D V_*}{\nu} \times 10^3$	$F_{bo} = \frac{V_{m_c}^2}{d_*}$	COMMENTS
Rubicon		Scott & Gravlee (1968)														
	Rubicon Gorge	Ø	260,000	-	10.8	0.0226	71.6	101.0	6.6	0.151	-	7.20	0.090	650.00	-	Maximum depths used in computing τ_{o_c}.
	Rubicon Gorge		260,000	-	7.8	0.0290	63.9	115.7	8.2	0.122	-	7.71	0.142	500.0	-	
	Rubicon Gorge		260,000	-	4.7	0.0179	64.1	71.6	13.6	0.074	-	6.07	0.145	238.0	-	D = average of largest material.
	Rubicon Gorge		260,000	-	6.0	0.0169	61.1	64.5	10.2	0.098	-	5.76	0.102	288.0	-	
	Fork of American River		260,000	-	1.6	0.0039	56.2	13.7	35.0	0.029	-	2.65	0.081	35.4	-	
	Oregon Bar		260,000	-	3.0	0.0036	53.1	11.9	17.7	0.056	-	2.48	0.038	62.0	-	
	Philadelphia Bar		260,000	-	1.6	0.0026	45.0	7.3	28.1	0.036	-	1.94	0.043	25.8	-	
	Diversion Dam		260,000	-	1.5	0.0030	36.2	6.8	24.1	0.042	-	1.87	0.043	23.5	-	
	Oregon Bar		260,000	-	2.1	0.0034	25.8	5.5	12.3	0.081	-	1.68	0.025	29.4	-	
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Truckee River		Birkeland (1968)														
	Verdi	▽	-	24.6	10.0	0.0070	60.	26.1	6.0	0.167	1.12	4.67	0.024	392.0	10.1	Velocities estimated by using Manning's equation with = 0.060.
	Truckee Meadows		-	7.4	3.0	0.0016	20.	2.0	6.6	0.150	0.34	1.01	0.0064	25.2	2.7	
	Mustang		-	13.1	10.0	0.0020	40.	5.0	4.0	0.250	0.32	1.61	0.0048	134.0	4.3	D = average of largest material.
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...

TABLE A-5

RIVER DATA - THRESHOLD OF MOTION OF BED-MATERIAL

RIVER	LOCATION	REFERENCE & PLOTTING SYMBOL	MAXIMUM DISCHARGE Q (cfs)	MEAN CRITICAL VELOCITY Vm_c (ft/sec)	PARTICLE SIZE D	SLOPE	MEAN DEPTH OF FLOW d_* (ft)	CRITICAL SHEAR STRESS τo_c (lb/ft²)	d_*/D	D/d_*	$\frac{\rho\, Vm_c^2}{\gamma_s'\, D}$	SHEAR VELOCITY V_* (ft/sec)	$\frac{\rho\, V_*^2}{\gamma_s'\, D}$	$\frac{D\, V_*}{\nu} \times 10^4$	$Fbo = \frac{Vm_c^2}{d_*}$	COMMENTS
Middle Fork Eel River (California)		Ritter (1967) □														
	#140		3,750	1.9	0.03	0.00755	2.1	1.0	71.5	0.014	2.22	0.714	0.320	0.178	1.7	Stations located along a cross-section at a gauging station.
	#150		3,750	4.1	0.19	0.00755	3.1	1.5	16.4	0.061	1.63	0.868	0.075	1.37	5.4	
	#160		3,750	4.7	0.11	0.00755	3.1	1.5	28.6	0.035	5.45	0.868	0.143	0.80	7.1	D = intermediate axis of largest material.
	#170		3,750	5.5	0.17	0.00755	3.8	1.8	22.2	0.045	3.30	0.960	0.073	1.36	8.0	
	#180		3,750	5.8	0.11	0.00755	4.2	2.0	38.4	0.026	5.65	1.010	0.174	1.00	8.0	
	#240		3,750	7.5	0.27	0.00755	3.6	1.7	13.3	0.075	3.85	0.935	0.060	2.10	15.6	
	#250		3,750	6.5	0.30	0.00755	3.1	1.5	10.3	0.097	2.60	0.868	0.048	2.17	13.6	
	#260		3,750	6.7	0.40	0.00755	2.8	1.3	7.0	0.143	2.06	0.820	0.031	2.75	16.0	
	#270		3,750	5.5	0.08	0.00755	2.8	1.3	34.5	0.029	7.00	0.820	0.155	0.55	10.8	
	#280		3,750	6.2	0.15	0.00755	3.1	1.5	20.8	0.048	4.72	0.868	0.095	1.08	12.4	
	#300		3,750	4.8	0.21	0.00755	3.3	1.6	15.6	0.064	2.04	0.895	0.073	1.56	7.0	
	#320		3,750	2.5	0.14	0.00755	0.9	0.4	6.4	0.156	0.82	0.468	0.027	0.55	7.0	
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Coffee Creek	Trinity County	Stewart & LaMarche (1967) Ø	-	15.0	2.9	-	-	-	-	-	-	-	-	-	-	
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Neri River Japan		Oishi (1956) ●	2,100	12.0	1.5	0.027	3.6	6.0	2.4	0.417	1.77	1.760	0.038	22.0	33.4	Velocity computed from Vm = Q/A
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...

TABLE A-5

RIVER DATA - THRESHOLD OF MOTION OF BED-MATERIAL

RIVER	LOCATION	REFERENCE & PLOTTING SYMBOL	MAXIMUM DISCHARGE Q (cfs)	MEAN CRITICAL VELOCITY Vm_c (ft/sec)	PARTICLE SIZE D	SLOPE	MEAN DEPTH OF FLOW d_* (ft)	CRITICAL SHEAR STRESS τo_c (lb/ft²)	d_*/D	D/d_*	$\frac{\rho Vm_c^2}{\gamma_s' D}$	SHEAR VELOCITY v_* (ft/sec)	$\frac{\rho v_*^2}{\gamma_s' D}$	$\frac{D v_*}{\nu} \times 10^4$	$Fbo = \frac{Vm_c^2}{d_*}$	COMMENTS
White River		Fahnestock (1963)														
		●	-	3.47	0.20	0.045	0.58	1.63	2.9	0.345	1.11	0.917	0.073	1.53	20.8	D = average of 5 largest boulders.
			-	5.80	0.36	0.025	1.30	2.04	3.6	0.277	1.72	1.020	0.054	3.06	25.8	
			-	3.76	0.20	0.018	0.72	0.81	3.6	0.277	1.30	0.646	0.039	1.08	19.7	Samples caught on screen.
			-	5.60	0.60	0.019	1.01	1.20	1.7	0.595	0.97	0.784	0.032	3.90	31.0	
			-	5.72	0.56	0.026	1.18	1.91	4.5	0.220	1.08	0.994	0.033	4.63	27.8	
			-	5.95	0.20	0.027	1.10	1.85	5.5	0.180	3.28	0.980	0.088	1.63	32.2	
			-	5.55	0.65	0.180	1.07	1.24	1.7	0.608	0.88	0.790	0.018	4.26	28.8	
			-	4.66	0.50	0.017	1.01	1.07	2.0	0.500	0.80	0.740	0.021	3.09	21.5	
			-	-	0.27	0.012	-	0.79	-	-	-	0.640	0.028	1.44	-	
			-	-	0.25	0.012	-	0.57	-	-	-	0.540	0.022	1.12	-	
			-	-	0.28	0.012	-	0.53	-	-	-	0.522	0.018	1.22	-	
			-	-	0.26	0.023	0.70	1.01	2.6	0.384	-	0.718	0.038	1.56	-	
			-	-	0.23	0.029	0.66	1.19	2.9	0.345	-	0.784	0.079	1.50	-	
			-	-	0.60	0.033	0.87	1.79	1.45	0.690	-	0.960	0.046	4.81	-	
			-	-	0.70	0.011	0.94	0.64	1.34	0.745	-	0.576	0.014	3.35	-	
			-	-	0.31	0.029	1.13	2.04	3.6	0.278	-	1.025	0.063	2.65	-	)
			-	-	1.00	0.056	1.10	3.85	1.1	0.910	-	1.410	0.037	11.80	-	)
			-	-	1.26	0.043	2.00	5.36	1.6	0.625	-	1.660	0.041	17.50	-	) Samples on bars.
			-	-	1.00	0.031	1.30	2.52	1.3	0.770	-	1.140	0.024	9.50	-	)
			-	-	1.12	0.019	1.30	1.54	1.16	0.861	-	0.890	0.013	8.33	-	
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...

TABLE A-5

RIVER DATA - THRESHOLD OF MOTION OF BED-MATERIAL

RIVER	LOCATION	REFERENCE & PLOTTING SYMBOL	MAXIMUM DISCHARGE Q (cfs)	MEAN CRITICAL VELOCITY Vm_c (ft/sec)	PARTICLE SIZE D	SLOPE	MEAN DEPTH OF FLOW d_* (ft)	CRITICAL SHEAR STRESS τo_c (lb/ft²)	d_*/D	D/d_*	$\frac{\rho Vm_c^2}{\gamma_s D}$	SHEAR VELOCITY V_* (ft/sec)	$\frac{\rho V_*^2}{\gamma_s' D}$	$\frac{D V_*}{\nu} \times 10^4$	$Fbo = \frac{Vm_c^2}{d_*}$	COMMENTS
Western Alberta Rivers		Van der Giessen (1966)														
Elbow River	1 mile U/S Bragg Creek	△	601	-	0.15	0.007	1.10	0.48	7.4	0.135	-	0.498	0.030	0.63	-	D = D_{90} by grid and number.
Drywood Creek	Sta. 27+00		446	-	0.24	0.012	2.60	1.95	10.8	0.093	-	1.000	0.077	2.00	-	
Rhone River	Pierre Benite Chute	Ramette & Heuzel (1962)														
			95,400	-	0.16	0.000325	22.4	0.45	140	0.007	-	0.481	0.027	0.65	-	
			113,000	-	0.25	0.000342	26.5	0.54	106	0.009	-	0.528	0.021	1.10	-	
			53,000	-	0.08	0.000325	19.2	0.39	240	0.004	-	0.448	0.046	0.30	-	
Elbow River	Bragg Creek	Hollingshead (1968)														
			800	2.4	0.25	0.00745	1.7	0.79	6.8	0.47	0.43	0.640	0.030	1.33	3.4	D = D_{90} by number.
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Connecticut Rivers		Wolman & Eiler (1958)														
Naugatuck R.	East Branch		6,210	9.0	5.0	0.0500	10.0	31.2	2.0	0.50	0.30	4.010	0.060	167.0	8.1	D = largest material moved.
Farmington R.			140,000	-	2.0	0.00012	25	0.2	12.5	0.08	-	0.316	0.001	5.26	-	
Farmington R. West Branch	Riverton		216	-	1.0	0.02	14	1.8	14.0	0.072	-	0.950	0.017	7.9	-	
Thorp Brook			12,000	-	1.3	0.067	5	21.0	3.8	0.263	-	3.29	0.153	35.7	-	
Farmington R. West Branch	New Boston		35,000	11.0	2.7	0.007	11	4.8	4.1	0.246	0.83	1.57	0.017	35.3	11.0	
Salmon Brook West Branch	West Granby 1.5 mi. NW		5,000	7.0	2.5	0.025	11	17.2	4.4	0.227	0.36	2.97	0.066	61.5	4.5	
Quinebaug R.	Quinebaug		49,300	7.0	1.0	0.002	10	1.2	10.0	0.100	0.90	0.80	0.012	6.7	4.9	
Salmon Brook West Branch	Granby		8,250	-	0.4	0.010	10	6.2	25.0	0.040	-	2.50	0.148	8.3	-	
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...

TABLE A-5

RIVER DATA - THRESHOLD OF MOTION OF BED-MATERIAL

RIVER	LOCATION	REFERENCE & PLOTTING SYMBOL	MAXIMUM DISCHARGE Q (cfs)	MEAN CRITICAL VELOCITY Vm_c (ft/sec)	PARTICLE SIZE D	SLOPE	MEAN DEPTH OF FLOW d_* (ft)	CRITICAL SHEAR STRESS τo_c (lb/ft^2)	d_*/D	D/d_*	$\frac{\rho Vm_c^2}{\gamma_s D}$	SHEAR VELOCITY v_* (ft/sec)	$\frac{\rho v_*^2}{\gamma_s' D}$	$\frac{D v_*}{\nu}$ $\times 10^4$	$Fbo = \frac{Vm_c^2}{d_*}$	COMMENTS
British Columbia Rivers		Kellerhals (1963)														
Chilko R.	Henry's Crossing	◇	5,600	8.1	0.83	0.00503	4.6	1.44	5.5	0.182	1.46	0.863	0.017	6.0	14.2	$D = D_{90}$ by number and taped grid.
Taseko R.	Taseko Lake		6,400	8.0	1.00	0.00290	5.3	0.96	5.3	0.189	1.18	0.710	0.010	5.9	12.0	
Chilko R.	Chilko Lake		7,000	4.0	0.58	0.00092	6.5	0.37	11.2	0.089	0.51	0.436	0.006	2.1	2.5	
Caribou R.	Quesnal Forks		12,000	9.6	1.50	0.00419	6.6	1.72	4.4	0.227	1.13	0.942	0.011	11.8	14.0	
Quesnal R.	Lawless Creek		14,800	11.4	1.30	0.00633	6.5	2.56	5.0	0.200	1.85	1.150	0.019	12.4	20.0	
Caribou R.	Caribou Lake		17,000	6.1	1.10	0.00192	10.8	1.30	9.8	0.102	0.62	0.818	0.011	7.5	3.4	
Thompson R.	Kamloops Lake		135,000	11.8	0.88	0.00072	25.6	1.14	28.2	0.035	2.92	0.766	0.012	5.6	5.4	
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Kellerhals Flume		Kellerhals (1963)														
Test 2		◆	1.00	2.10	0.0459	0.00990	0.183	0.113	4.00	0.250	1.78	0.766	0.023	0.29	24.0	$D = D_{90}$ by weight.
Test 3			1.00	2.27	0.0459	0.01240	0.147	0.114	3.20	0.313	2.08	0.768	0.024	0.29	35.0	
Test 4			2.25	2.57	0.0459	0.00625	0.258	0.100	5.63	0.178	2.65	0.718	0.021	0.27	25.6	
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...

TABLE A-5

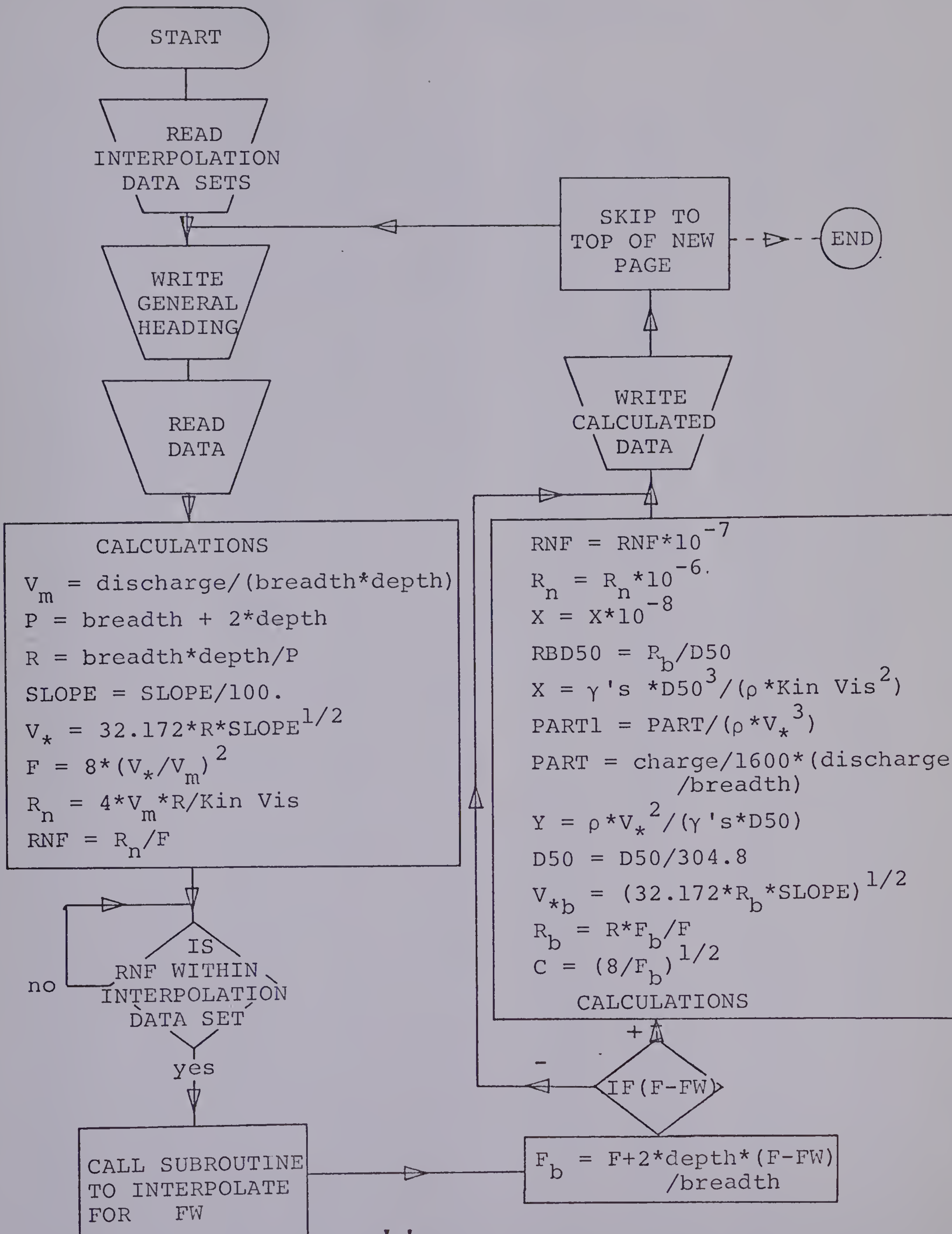
RIVER DATA - THRESHOLD OF MOTION OF BED-MATERIAL

RIVER	LOCATION	REFERENCE & PLOTTING SYMBOL	MAXIMUM DISCHARGE Q (cfs)	MEAN CRITICAL VELOCITY Vm_c (ft/sec)	PARTICLE SIZE D	SLOPE	MEAN DEPTH OF FLOW d_* (ft)	CRITICAL SHEAR STRESS τo_c (lb/ft²)	d_*/D	D/d_*	$\frac{\rho Vm_c^2}{\gamma_s' D}$	SHEAR VELOCITY v_* (ft/sec)	$\frac{\rho v_*^2}{\gamma_s' D}$	$\frac{D v_*}{\nu} \times 10^4$	$Fbo = \frac{Vm_c^2}{d_*}$	COMMENTS
San Luis Valley Canals	Sec. 1	Lane & Carlson (1953) X	1,500	5.88	0.70	0.00280	3.50	0.625	5.00	0.200	0.92	0.567	0.009	3.31	9.74	$D = D_{90}$ by weight.
	2		668	5.83	0.50	0.00376	2.08	0.488	4.16	0.240	1.26	0.502	0.009	2.09	16.32	
	4		768	6.53	0.53	0.00359	2.54	0.570	4.80	0.208	1.49	0.542	0.010	2.39	16.80	
	5		448	5.82	0.28	0.00368	1.92	0.441	6.86	0.146	2.24	0.477	0.015	1.12	17.65	
	6		159	4.59	0.20	0.00295	1.60	0.294	8.00	0.125	1.95	0.389	0.014	0.65	13.20	
	7		95.6	4.36	0.18	0.00290	1.38	0.250	7.68	0.130	1.95	0.359	0.013	0.54	13.78	
	8		46	3.00	0.26	0.00316	0.80	0.158	3.07	0.326	0.64	0.286	0.006	0.62	11.22	
	10		16.6	2.90	0.40	0.00965	0.52	0.314	1.30	0.770	0.11	0.404	0.008	1.35	16.17	
	11		203	3.88	0.24	0.00235	1.62	0.238	6.75	0.148	1.16	0.350	0.010	0.70	9.30	
	12		128	4.00	0.17	0.00243	1.46	0.222	8.58	0.116	1.74	0.338	0.012	0.48	10.96	
	14		110	3.29	0.12	0.00136	1.56	0.132	13.00	0.077	1.67	0.261	0.011	0.26	6.95	
	15		477	4.84	0.28	0.00199	2.50	0.310	8.93	0.112	1.55	0.400	0.011	0.93	9.36	
	17		531	5.51	0.17	0.00274	2.35	0.402	13.82	0.072	3.30	0.455	0.023	0.65	12.92	
	18		235	3.80	0.10	0.00080	2.48	0.124	24.80	0.040	2.67	0.253	0.012	0.21	5.83	
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...

APPENDIX 4

FLUME DATA

COARSE BED MATERIAL



NOTE: '*' means multiply except where it is used as a subscript

COMPUTER PROGRAM- BED-LOAD TRANSPORT

FIGURE A-10

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

GILBERT - GRADE E, 1.71MM, UNIFORM

TEST	VMEAN	F	C	RB	V*R	Y	PART1	RB/D50
1- 1	2.21	0.058	10.661	0.111	0.207	0.141	4.412	19.3
1- 2	2.26	0.051	11.486	0.103	0.205	0.139	4.524	18.4
1- 3	2.40	0.055	11.052	0.102	0.217	0.155	4.958	18.2
1- 4	2.40	0.056	10.843	0.103	0.221	0.161	4.929	18.3
1- 5	2.42	0.058	10.683	0.102	0.226	0.169	4.515	18.2
1- 6	2.75	0.044	11.701	0.165	0.235	0.182	4.709	29.4
1- 7	2.79	0.052	10.682	0.167	0.261	0.225	4.367	29.8
1- 8	2.75	0.053	10.470	0.170	0.263	0.227	4.499	30.3
1- 9	2.97	0.045	11.735	0.155	0.253	0.211	5.122	27.6
1-10	2.99	0.052	9.515	0.298	0.314	0.325	2.645	53.1
1-11	2.87	0.061	8.612	0.318	0.334	0.367	2.275	56.6
1-12	3.02	0.033	11.627	0.374	0.259	0.222	4.659	66.6
1-13	1.94	0.060	10.939	0.088	0.177	0.103	3.912	15.6
1-14	2.09	0.049	12.276	0.081	0.170	0.096	4.384	14.4
1-15	2.25	0.064	10.677	0.076	0.210	0.146	5.536	13.6
1-16	2.36	0.055	11.545	0.072	0.205	0.138	6.217	12.9
1-17	1.96	0.102	8.306	0.089	0.236	0.183	4.348	15.9
1-18	1.02	0.036	12.057	0.269	0.078	0.020	0.025	47.9
1-20	1.31	0.060	9.966	0.239	0.131	0.057	1.234	42.6
1-21	1.50	0.047	11.686	0.205	0.128	0.054	1.161	36.5
1-22	1.69	0.061	10.163	0.191	0.166	0.091	1.786	34.0
1-23	18.15	0.001	0.000	0.000	0.000	0.000	0.000	0.0
1-24	2.17	0.057	10.809	0.151	0.201	0.133	3.207	26.8
1-25	2.21	0.043	12.892	0.103	0.249	0.204	7.331	18.3
1-26	3.16	0.055	11.295	0.106	0.279	0.257	8.037	19.0
1-27	1.19	0.020	18.730	0.312	0.063	0.013	0.092	55.7
1-28	1.31	0.028	14.468	0.362	0.090	0.027	0.032	64.5
1-29	1.64	0.043	11.242	0.349	0.145	0.070	0.691	62.2
1-30	1.34	0.071	8.026	0.457	0.167	0.092	0.460	81.5
1-31	1.82	0.052	10.168	0.332	0.179	0.106	1.127	59.3
1-32	1.80	0.051	10.310	0.334	0.175	0.101	1.811	59.5
1-33	2.02	0.059	9.606	0.309	0.210	0.146	2.485	55.2
1-34	2.42	0.040	12.373	0.248	0.196	0.126	3.088	44.2
1-35	2.44	0.044	11.675	0.251	0.209	0.144	3.114	44.8
1-36	2.27	0.072	8.735	0.286	0.259	0.221	2.793	51.0
1-37	2.25	0.074	8.536	0.289	0.262	0.226	2.767	51.5
1-38	2.69	0.052	10.703	0.236	0.251	0.208	3.297	42.1
1-39	3.11	0.051	11.053	0.207	0.281	0.261	4.458	36.9
1-40	3.29	0.044	12.051	0.193	0.273	0.246	5.454	34.4
1-41	3.93	0.034	14.239	0.160	0.276	0.250	7.451	28.4
1-42	3.88	0.035	13.995	0.162	0.277	0.254	7.746	28.8
1-43	2.94	0.033	13.283	0.294	0.222	0.162	5.062	52.4
1-44	2.76	0.045	11.000	0.331	0.251	0.207	3.506	59.0
1-45	2.10	0.043	13.057	0.120	0.161	0.085	4.157	21.4
1-46	2.13	0.045	12.774	0.119	0.167	0.092	3.717	21.1
1-47	2.84	0.048	12.401	0.091	0.229	0.172	6.413	16.3
1-48	2.33	0.050	11.466	0.213	0.203	0.136	3.409	38.0
1-49	2.81	0.055	11.097	0.181	0.253	0.211	4.576	32.2

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

GILBERT - GRADE F, 1.71MM, UNIFORM

TEST	VMEAN	F	C	PB	V*B	Y	PART1	PB/D50
1-50	2.64	0.066	9.942	0.195	0.265	0.231	3.991	34.7
1-51	2.62	0.047	11.417	0.281	0.222	0.173	3.426	50.2
1-52	2.57	0.048	11.301	0.276	0.235	0.182	3.314	49.3

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

GILBERT - GRADE F, 3.17MM, UNIFORM

TEST	VMEAN	F	C	RB	Y*B	Y	PART1	RB/D50
1- 1	2.34	0.053	11.230	0.104	0.203	0.077	2.100	10.0
1- 2	2.14	0.068	9.673	0.116	0.221	0.087	1.836	11.1
1- 3	2.70	0.069	9.842	0.093	0.275	0.134	4.587	9.0
1- 4	2.60	0.076	9.288	0.098	0.290	0.139	4.326	9.4
1- 5	2.66	0.052	10.591	0.175	0.251	0.112	2.839	16.8
1- 6	2.70	0.050	10.791	0.172	0.250	0.111	2.874	16.5
1- 7	3.13	0.057	10.235	0.153	0.305	0.166	4.545	14.7
1- 8	3.24	0.054	10.603	0.148	0.305	0.165	4.304	14.2
1- 9	3.21	0.041	11.172	0.265	0.288	0.147	2.603	25.5
1-10	3.27	0.040	11.298	0.261	0.290	0.149	2.584	25.0
1-11	2.02	0.065	10.512	0.085	0.192	0.066	1.087	8.1
1-12	2.33	0.079	9.554	0.074	0.244	0.106	3.241	7.2
1-13	2.27	0.097	9.116	0.077	0.250	0.111	3.112	7.4
1-14	2.27	0.052	11.451	0.144	0.198	0.070	1.429	13.9
1-15	2.25	0.056	10.928	0.145	0.206	0.076	1.291	14.0
1-16	2.84	0.054	11.359	0.117	0.250	0.111	3.577	11.3
1-17	2.79	0.058	10.913	0.120	0.256	0.116	3.463	11.5
1-18	2.59	0.070	9.784	0.130	0.265	0.125	3.267	12.5
1-19	2.74	0.046	11.507	0.229	0.233	0.101	2.157	22.0
1-20	2.77	0.045	11.720	0.225	0.225	0.099	2.204	21.7
1-21	3.39	0.052	11.034	0.192	0.307	0.167	3.981	18.5
1-22	3.49	0.050	11.215	0.187	0.310	0.171	4.160	18.0
1-23	3.39	0.036	12.975	0.265	0.261	0.121	3.352	25.5
1-24	3.26	0.042	11.737	0.283	0.278	0.137	2.871	27.2
1-25	2.37	0.052	11.783	0.108	0.201	0.072	2.219	10.4
1-26	2.32	0.057	11.222	0.111	0.209	0.077	2.018	10.7
1-27	2.41	0.088	8.982	0.109	0.268	0.128	3.071	10.5
1-28	2.55	0.076	9.716	0.103	0.262	0.122	3.396	9.9
1-29	2.62	0.050	11.609	0.191	0.226	0.091	1.957	18.4
1-30	2.66	0.049	11.720	0.188	0.227	0.092	1.960	18.1
1-31	2.59	0.054	11.136	0.195	0.232	0.096	1.831	18.7
1-32	3.16	0.053	11.380	0.162	0.278	0.137	4.185	15.6
1-33	3.09	0.060	10.609	0.167	0.291	0.151	3.664	16.0
1-34	3.08	0.039	12.974	0.227	0.238	0.100	3.462	22.8
1-35	2.94	0.050	11.216	0.255	0.262	0.122	2.569	24.5
1-36	2.98	0.048	11.470	0.251	0.260	0.120	2.913	24.1

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

GILBERT - GRADE G, 4.94MM, UNIFORM

TEST	VMEAN	F	C	RB	V*B	Y	PART1	RB/D50
1- 1	2.73	0.046	11.452	0.165	0.243	0.067	1.266	10.2
1- 2	2.86	0.050	10.916	0.158	0.271	0.084	2.140	9.8
1- 3	2.95	0.056	10.209	0.165	0.281	0.090	2.086	10.2
1- 4	3.14	0.054	10.550	0.152	0.293	0.101	3.184	9.4
1- 5	3.14	0.057	10.299	0.152	0.305	0.106	3.030	9.4
1- 6	3.44	0.060	10.069	0.142	0.341	0.133	4.334	8.7
1- 7	3.49	0.061	9.976	0.140	0.349	0.139	4.170	8.7
1- 8	2.93	0.034	12.311	0.268	0.242	0.067	1.122	16.5
1- 9	3.05	0.033	12.617	0.260	0.242	0.067	1.274	16.1
1-10	3.25	0.039	11.553	0.259	0.281	0.090	1.976	16.0
1-11	3.45	0.042	11.123	0.251	0.310	0.110	2.820	15.5
1-12	3.43	0.042	11.015	0.254	0.312	0.111	2.783	15.7
1-13	3.81	0.047	10.542	0.237	0.361	0.149	3.836	14.6
1-14	4.26	0.050	10.326	0.218	0.413	0.194	5.048	13.4
1-15	4.21	0.050	10.333	0.220	0.408	0.190	5.446	13.6
1-16	3.04	0.026	10.978	0.384	0.277	0.097	0.894	23.7
1-17	3.69	0.036	11.435	0.330	0.322	0.118	2.577	20.3
1-18	3.76	0.035	11.536	0.324	0.326	0.121	2.589	20.0
1-19	4.10	0.038	11.135	0.309	0.365	0.153	3.405	19.1
1-20	4.12	0.037	11.493	0.303	0.359	0.147	3.980	18.7
1-21	4.53	0.043	10.520	0.293	0.431	0.212	4.450	18.1
1-22	4.49	0.044	10.396	0.297	0.431	0.212	4.489	18.3
1-23	4.79	0.045	10.240	0.283	0.468	0.249	3.502	17.5
1-24	2.54	0.056	10.994	0.130	0.231	0.061	0.921	8.1
1-25	2.61	0.061	10.587	0.128	0.247	0.069	1.558	7.9
1-26	2.67	0.062	10.472	0.125	0.255	0.074	1.716	7.7
1-27	2.57	0.069	9.866	0.131	0.261	0.078	1.711	8.1
1-28	2.75	0.063	10.462	0.122	0.263	0.079	2.125	7.5
1-29	2.81	0.070	9.696	0.120	0.284	0.092	2.543	7.4
1-30	3.18	0.065	10.383	0.107	0.307	0.107	3.955	6.6
1-31	2.70	0.049	11.152	0.233	0.242	0.067	0.859	14.4
1-32	2.96	0.042	12.257	0.211	0.241	0.066	1.612	13.0
1-33	2.96	0.046	11.581	0.214	0.256	0.074	1.700	13.2
1-34	2.92	0.049	11.226	0.217	0.261	0.077	1.672	13.4
1-35	3.12	0.054	10.733	0.207	0.291	0.097	2.303	12.8
1-36	3.43	0.055	10.641	0.191	0.322	0.118	3.389	11.8
1-37	3.85	0.055	10.854	0.171	0.356	0.144	4.919	10.6
1-38	3.84	0.057	10.590	0.173	0.363	0.150	4.948	10.7
1-39	2.39	0.084	9.225	0.110	0.259	0.077	1.211	6.8
1-40	2.64	0.065	10.560	0.099	0.250	0.072	1.400	6.1
1-41	2.55	0.083	9.305	0.103	0.274	0.085	2.119	6.4
1-42	2.62	0.080	9.520	0.101	0.275	0.086	2.042	6.2
1-43	2.96	0.072	10.044	0.089	0.294	0.099	3.264	5.5
1-44	2.84	0.084	9.293	0.093	0.305	0.106	3.339	5.8
1-45	2.83	0.044	11.258	0.317	0.256	0.074	0.681	19.6
1-46	3.13	0.036	12.764	0.284	0.246	0.069	1.974	17.5
1-47	3.12	0.037	12.531	0.287	0.249	0.071	2.272	17.7
1-48	3.45	0.041	11.936	0.268	0.289	0.095	2.396	16.6

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

GILBERT - GRADE G, 4.94MM, UNIFORM

TEST	VMEAN	F	C	RB	V*B	Y	PART1	RB/D50
1-49	3.43	0.039	12.346	0.267	0.273	0.088	2.751	16.5
1-50	3.63	0.045	11.357	0.261	0.320	0.117	3.501	16.1
1-51	3.64	0.043	10.960	0.262	0.333	0.126	3.156	16.2
1-52	4.07	0.049	11.010	0.239	0.370	0.156	4.714	14.7
1-53	4.44	0.045	11.732	0.218	0.373	0.163	5.328	13.5
1-54	4.42	0.049	11.152	0.221	0.397	0.179	5.606	13.7
1-55	2.65	0.043	11.362	0.189	0.223	0.057	0.741	11.7
1-56	2.65	0.048	11.862	0.189	0.223	0.057	0.854	11.7
1-57	2.97	0.051	11.573	0.177	0.248	0.070	1.392	10.9
1-58	2.73	0.058	10.716	0.184	0.259	0.077	1.282	11.3
1-59	2.89	0.065	10.112	0.179	0.285	0.093	1.898	11.0
1-60	3.25	0.050	10.559	0.159	0.305	0.106	3.091	9.8
1-61	3.41	0.071	9.834	0.153	0.347	0.137	4.119	9.5
1-62	2.85	0.046	11.725	0.259	0.243	0.068	0.719	16.0
1-63	3.14	0.045	11.970	0.228	0.262	0.078	2.315	14.7
1-64	3.25	0.049	11.480	0.233	0.283	0.091	2.243	14.4
1-65	3.42	0.052	11.142	0.223	0.307	0.107	2.837	13.8
1-66	3.46	0.054	10.954	0.222	0.316	0.114	2.860	13.7
1-67	3.62	0.059	10.457	0.211	0.352	0.141	2.878	13.0
1-68	4.22	0.048	11.800	0.183	0.357	0.146	5.444	11.3
1-69	4.20	0.051	11.454	0.195	0.366	0.153	5.330	11.4

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

GILBERT - GRADE H, 7.01MM, UNIFORM

TEST	VMEAN	F	C	RB	V*B	Y	PART1	RB/D50
1- 1	2.99	0.051	10.895	0.157	0.274	0.060	0.798	6.8
1- 2	2.99	0.054	10.539	0.158	0.284	0.065	0.821	6.9
1- 3	3.01	0.060	9.881	0.160	0.304	0.074	1.165	6.9
1- 4	3.19	0.053	10.632	0.150	0.298	0.071	1.405	6.5
1- 5	3.29	0.064	9.681	0.148	0.340	0.093	2.250	6.4
1- 6	3.18	0.071	9.072	0.155	0.359	0.099	2.162	6.7
1- 7	3.22	0.038	11.778	0.259	0.274	0.060	0.757	11.2
1- 8	3.20	0.041	11.207	0.266	0.285	0.065	0.782	11.6
1- 9	3.34	0.042	11.033	0.259	0.303	0.074	1.279	11.3
1-10	3.23	0.049	9.949	0.276	0.325	0.085	1.086	12.0
1-11	3.43	0.052	9.730	0.262	0.355	0.101	1.996	11.4
1-12	3.59	0.048	10.272	0.251	0.349	0.098	2.102	10.9
1-13	3.85	0.054	9.721	0.241	0.396	0.126	2.917	10.5
1-14	4.45	0.050	10.449	0.209	0.426	0.146	4.689	9.1
1-15	4.40	0.056	9.773	0.215	0.450	0.163	4.095	9.4
1-16	3.33	0.033	11.798	0.344	0.286	0.066	0.815	15.0
1-17	3.32	0.039	10.708	0.364	0.308	0.076	0.588	15.8
1-18	3.37	0.040	10.394	0.367	0.324	0.084	0.561	16.0
1-19	3.61	0.039	10.655	0.346	0.323	0.092	1.154	15.0
1-20	3.84	0.042	10.429	0.324	0.368	0.109	1.800	14.5
1-21	3.88	0.041	10.531	0.330	0.368	0.109	1.824	14.3
1-22	3.79	0.045	9.888	0.344	0.384	0.118	1.616	15.0
1-23	4.34	0.040	10.879	0.299	0.397	0.128	2.761	13.0
1-24	4.26	0.042	10.633	0.306	0.401	0.129	2.803	13.3
1-25	4.33	0.040	10.957	0.299	0.395	0.125	3.055	13.0
1-26	4.99	0.040	11.220	0.266	0.444	0.159	4.106	11.6
1-27	5.08	0.039	11.355	0.261	0.447	0.161	4.035	11.3

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

MEYER-PETER&MULLER- 28.65MM, UNIFORM

TEST	VMEAN	F	C	RB	Y*R	Y	PART1	RB/D50
1- 1	6.95	0.052	10.919	1.181	0.638	0.080	0.761	12.6
1- 2	6.07	0.046	11.462	1.332	0.529	0.055	0.035	14.2
1- 3	6.78	0.049	11.257	1.204	0.597	0.070	0.506	12.8
1- 4	6.75	0.049	11.237	1.210	0.601	0.071	0.492	12.9
1- 5	6.75	0.049	11.269	1.210	0.599	0.071	0.494	12.9
1- 6	6.36	0.057	10.297	1.294	0.617	0.075	0.469	13.8
1- 7	6.77	0.048	11.370	1.205	0.596	0.070	0.509	12.8
1- 8	7.31	0.058	10.330	1.135	0.708	0.099	1.173	12.1
1- 9	6.52	0.043	11.374	1.247	0.574	0.065	0.287	13.3
1-10	6.44	0.046	11.597	1.259	0.555	0.061	0.203	13.4
1-11	5.92	0.042	11.999	1.351	0.494	0.048	0.028	14.4
1-12	7.37	0.062	10.121	1.061	0.777	0.119	1.039	11.3
1-13	8.09	0.048	10.763	1.777	0.835	0.137	1.583	18.9
1-14	8.22	0.042	11.492	1.905	0.721	0.102	1.116	20.3
1-15	7.73	0.041	11.437	2.010	0.676	0.090	0.674	21.4
1-16	7.44	0.038	11.871	2.064	0.626	0.077	0.427	22.0
1-17	7.36	0.039	11.834	2.082	0.622	0.076	0.436	22.2
1-18	6.43	0.033	12.595	2.302	0.511	0.051	0.026	24.5
1-19	6.27	0.034	12.290	2.361	0.512	0.052	0.021	25.1
1-20	8.95	0.049	10.587	1.789	0.845	0.140	1.525	19.0
1-21	9.45	0.046	10.449	2.338	0.904	0.161	1.242	24.9
1-22	9.42	0.045	10.653	2.276	0.884	0.154	1.339	24.2
1-23	8.73	0.038	11.491	2.435	0.760	0.114	0.945	25.9
1-24	8.76	0.037	11.596	2.410	0.756	0.112	0.955	25.6
1-25	8.28	0.034	12.115	2.513	0.694	0.092	0.657	26.7
1-26	9.17	0.035	11.854	2.569	0.689	0.093	0.637	27.3
1-27	7.75	0.033	12.098	2.665	0.641	0.081	0.397	28.4
1-28	7.36	0.034	12.032	2.640	0.653	0.084	0.372	28.1
1-29	7.42	0.031	12.573	2.731	0.590	0.068	1.375	29.1
1-30	7.36	0.032	12.330	2.767	0.597	0.070	1.260	29.4
1-31	6.91	0.029	12.761	2.876	0.542	0.058	0.020	30.6
1-32	6.91	0.030	12.570	2.892	0.550	0.059	0.019	30.8
1-33	7.68	0.066	9.749	1.090	0.787	0.122	1.891	11.6
1-34	6.52	0.035	12.173	2.295	0.536	0.056	0.022	24.4

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

MEYER-PETER&MULLER- 5.21MM, UNIFORM

TEST	VMEAN	F	C	RB	V*B	Y	PART1	RB/D50
2- 1	3.35	0.076	9.163	0.183	0.366	0.145	1.879	10.7
2- 2	2.83	0.051	11.180	0.207	0.254	0.069	0.490	12.1
2- 3	2.60	0.047	11.689	0.222	0.222	0.053	0.066	13.0
2- 4	3.02	0.057	10.619	0.198	0.284	0.087	1.016	11.6
2- 5	3.30	0.062	10.253	0.183	0.322	0.112	1.632	10.7
2- 6	3.49	0.067	9.877	0.175	0.354	0.135	2.109	10.2
2- 7	4.14	0.042	11.337	0.370	0.365	0.144	1.946	21.7
2- 8	3.82	0.041	11.391	0.395	0.336	0.122	1.450	23.1
2- 9	3.52	0.036	12.289	0.410	0.286	0.089	1.006	24.0
2-10	3.53	0.036	12.196	0.410	0.289	0.090	0.978	24.0
2-11	3.37	0.040	12.014	0.323	0.280	0.085	10.741	18.9
2-12	3.23	0.033	12.655	0.432	0.255	0.070	0.504	25.3
2-13	2.85	0.031	13.098	0.467	0.219	0.052	0.068	27.3
2-14	2.89	0.031	13.104	0.461	0.221	0.053	0.067	26.9
2-15	2.77	0.033	13.239	0.365	0.209	0.047	0.079	21.4
2-16	3.99	0.045	11.545	0.284	0.345	0.129	2.309	16.6
2-17	3.09	0.037	12.413	0.342	0.247	0.067	0.540	20.1
2-18	4.19	0.028	11.417	0.470	0.367	0.146	1.919	27.5
2-19	3.58	0.034	11.906	0.521	0.300	0.097	0.871	30.5
2-20	3.07	0.029	12.321	0.559	0.239	0.062	0.171	32.7

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

MEYER-PETER&MULLER- 4.40MM, MIXTURE

TEST	VMEAN	F	C	RB	Y*R	Y	PARTI	RB/D50
3- 1	3.03	0.060	11.171	0.277	0.271	0.093	1.958	13.9
3- 2	3.35	0.059	11.242	0.344	0.293	0.113	2.738	23.6
3- 3	3.40	0.057	11.465	0.338	0.297	0.112	2.672	23.2
3- 4	3.87	0.044	13.021	0.335	0.296	0.111	3.342	23.0
3- 5	3.22	0.059	11.233	0.316	0.288	0.106	2.365	21.9
3- 6	2.63	0.076	9.960	0.275	0.269	0.091	1.510	18.6
3- 7	2.41	0.075	10.091	0.219	0.239	0.070	0.943	14.5

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

MEYER-PETER&MULLER- 3.30MM, MIXTURE

TEST	VMEAN	F	C	RB	Y*B	Y	PART1	RB/D50
4- 1	2.22	0.051	10.151	0.553	0.218	0.081	0.194	51.0
4- 2	2.15	0.052	10.146	0.517	0.212	0.077	0.168	47.8
4- 3	2.07	0.055	10.020	0.494	0.207	0.073	0.119	44.7
4- 4	0.63	0.926	1.776	1.449	0.357	0.218	0.006	132.9
4- 5	1.30	0.060	9.955	0.371	0.181	0.056	0.012	34.2
4- 6	1.62	0.073	9.095	0.336	0.179	0.055	0.003	31.0
4- 7	1.45	0.075	9.241	0.278	0.157	0.042	0.0	25.7
4- 8	1.23	0.033	8.865	0.237	0.145	0.036	0.0	21.9
4- 9	2.16	0.053	10.079	0.516	0.214	0.078	0.145	47.6
4-10	2.45	0.047	10.209	0.654	0.240	0.098	0.885	60.4
4-11	2.16	0.053	10.093	0.516	0.214	0.078	0.311	47.7
4-12	2.46	0.048	10.207	0.652	0.241	0.099	1.078	60.2
4-13	2.43	0.048	10.203	0.624	0.239	0.097	1.055	57.6
4-14	2.32	0.050	10.232	0.579	0.227	0.088	0.797	53.5
4-15	2.23	0.052	10.078	0.552	0.221	0.084	0.593	51.0
4-16	2.15	0.053	10.109	0.517	0.213	0.077	0.496	47.7
4-17	2.01	0.056	9.997	0.463	0.201	0.069	0.234	42.8
4-18	1.87	0.060	9.881	0.406	0.189	0.061	0.119	37.5
4-19	2.16	0.052	10.234	0.513	0.211	0.076	0.450	47.4
4-20	1.92	0.056	10.230	0.402	0.183	0.074	0.382	45.4
4-21	2.17	0.052	10.195	0.522	0.213	0.064	0.164	39.5
4-22	1.92	0.056	10.113	0.421	0.190	0.050	0.017	31.9
4-23	2.49	0.047	10.232	0.656	0.244	0.083	0.618	49.6
4-24	2.01	0.055	10.041	0.464	0.200	0.056	0.030	35.1

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

MEYER-PETER&MULLER- 2.00MM, MIXTURE

TEST	VMEAN	F	C	RB	Y*B	Y	PARTI	RB/D50
5- 1	2.20	0.042	13.353	0.351	0.165	0.090	2.119	63.0
5- 2	2.10	0.035	14.944	0.246	0.141	0.070	0.959	47.2
5- 3	2.09	0.036	14.616	0.248	0.143	0.070	1.055	46.4
5- 4	2.04	0.028	16.705	0.178	0.122	0.050	0.542	32.4
5- 5	2.08	0.041	13.626	0.298	0.153	0.080	1.437	55.4
5- 6	2.47	0.044	12.945	0.456	0.191	0.108	2.459	75.2
5- 7	2.40	0.042	13.273	0.385	0.181	0.099	2.441	63.1
5- 8	2.39	0.047	12.246	0.581	0.234	0.157	0.249	89.3
5- 9	2.56	0.050	11.824	0.632	0.225	0.143	1.794	96.3
5-10	2.66	0.049	12.003	0.631	0.221	0.140	2.120	97.7
5-11	2.62	0.050	11.310	0.640	0.222	0.140	2.059	98.6
5-12	2.73	0.048	12.104	0.615	0.225	0.140	2.427	91.8
5-13	2.74	0.049	12.064	0.613	0.227	0.141	2.190	90.7
5-14	2.58	0.045	12.724	0.514	0.203	0.117	2.484	79.2
5-15	2.51	0.044	12.839	0.527	0.195	0.108	2.080	80.9
5-16	2.14	0.046	12.819	0.362	0.167	0.085	1.333	59.9
5-17	2.31	0.044	12.920	0.422	0.179	0.088	1.726	63.1

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

BOGARDI&YEN- 10.0MM, UNIFORM

TEST	VMEAN	F	C	RB	Y*R	Y	PARTI	RB/D50
1- 1	3.18	0.090	9.015	0.166	0.352	0.070	0.034	5.1
1- 2	2.75	0.141	7.126	0.190	0.385	0.084	0.094	5.8
1- 3	2.83	0.138	7.193	0.194	0.393	0.087	0.138	5.9
1- 4	2.77	0.127	7.560	0.172	0.366	0.076	0.029	5.3
1- 5	2.73	0.097	8.492	0.247	0.322	0.058	0.045	7.5
1- 6	2.75	0.167	6.358	0.284	0.433	0.105	0.077	9.7
1- 7	2.70	0.166	6.424	0.265	0.421	0.100	0.030	8.1
1- 8	2.67	0.149	6.766	0.276	0.395	0.088	0.035	8.4
1- 9	2.74	0.128	7.060	0.265	0.383	0.085	0.030	8.1
1-10	2.62	0.163	6.434	0.294	0.407	0.093	0.027	9.0
1-11	2.71	0.148	6.776	0.283	0.399	0.090	0.054	8.6
1-12	2.71	0.154	6.604	0.295	0.410	0.095	0.076	9.0
1-13	2.81	0.139	6.974	0.286	0.402	0.091	0.059	8.7
1-14	2.75	0.128	7.424	0.225	0.371	0.077	0.039	6.9
1-15	2.87	0.123	7.386	0.241	0.383	0.085	0.117	7.4
1-16	2.91	0.119	7.749	0.201	0.375	0.079	0.154	6.1
1-17	3.44	0.069	9.369	0.210	0.367	0.076	0.530	6.4
1-18	2.55	0.109	7.142	0.255	0.356	0.072	0.025	7.8
1-19	3.25	0.056	10.047	0.274	0.324	0.059	0.014	8.4
1-20	3.35	0.069	8.435	0.371	0.397	0.089	0.031	11.3

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

BOGARDI&YEN- 6.8MM, UNIFORM

TEST	VMEAN	F	C	RB	Y*B	Y	PART1	RB/D50
2- 1	2.39	0.100	8.552	0.162	0.278	0.064	0.020	7.3
2- 2	2.29	0.089	9.141	0.129	0.244	0.049	0.005	5.9
2- 3	2.73	0.098	8.553	0.200	0.319	0.084	0.232	9.0
2- 4	2.54	0.095	8.912	0.150	0.288	0.069	0.048	6.7
2- 5	2.70	0.098	8.630	0.171	0.312	0.081	0.215	7.7
2- 6	2.51	0.099	8.717	0.129	0.288	0.068	0.045	5.8
2- 7	2.52	0.095	8.943	0.128	0.285	0.067	0.040	5.7
2- 8	2.95	0.094	8.854	0.160	0.322	0.086	0.430	7.2
2- 9	2.55	0.091	9.079	0.124	0.281	0.066	0.041	5.6
2-10	2.59	0.104	8.479	0.126	0.305	0.077	0.299	5.7
2-11	2.36	0.109	8.298	0.113	0.285	0.067	0.057	5.1
2-12	2.51	0.101	8.645	0.115	0.290	0.070	0.089	5.1
2-13	2.63	0.098	8.570	0.196	0.313	0.081	0.064	8.8
2-14	2.64	0.110	7.981	0.235	0.331	0.091	0.210	10.5
2-15	2.53	0.099	8.560	0.177	0.295	0.072	0.024	7.9
2-16	2.75	0.096	8.667	0.196	0.317	0.083	0.069	8.8
2-17	3.42	0.044	11.748	0.241	0.291	0.070	0.340	10.8
2-18	2.07	0.055	10.784	0.168	0.276	0.063	0.101	7.5

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

BOGARDI&YEN- 15.0MM, UNIFORM

TEST	VMEAN	F	C	RB	V*R	Y	PART1	RB/D50
3- 1	3.61	0.063	8.087	0.543	0.446	0.075	0.012	11.0
3- 2	3.74	0.062	8.874	0.385	0.421	0.067	0.005	7.8
3- 3	4.02	0.047	10.382	0.374	0.383	0.056	0.031	7.6
3- 4	4.09	0.040	11.605	0.344	0.352	0.047	0.072	7.0
3- 5	4.03	0.051	10.421	0.290	0.387	0.056	0.061	5.9
3- 6	4.29	0.056	9.752	0.324	0.440	0.073	0.088	6.6
3- 7	3.95	0.055	10.282	0.247	0.385	0.056	0.065	5.0
3- 8	4.48	0.051	10.545	0.269	0.425	0.068	0.418	5.5
3- 9	3.94	0.055	10.241	0.260	0.385	0.056	0.126	5.3
3-10	3.73	0.058	10.230	0.212	0.365	0.050	0.016	4.3

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

USFWS - 4.1MM, MIXTURE

TEST	VMEAN	F	C	RB	V*B	Y	PART1	RB/D50
9- 1	1.86	0.045	12.653	0.223	0.147	0.030	0.005	16.6
9- 2	1.99	0.043	12.356	0.247	0.154	0.033	0.023	19.4
9- 3	2.04	0.045	12.459	0.278	0.164	0.037	0.032	20.7
9- 4	2.14	0.044	12.398	0.309	0.173	0.041	0.046	23.0
9- 5	2.22	0.045	12.263	0.340	0.181	0.045	0.036	25.3
9- 6	2.16	0.053	10.986	0.401	0.197	0.053	0.382	29.8
9- 7	1.85	0.056	11.258	0.211	0.165	0.037	0.027	15.6
9- 8	2.00	0.053	11.494	0.235	0.174	0.042	0.062	17.5
9- 9	2.12	0.051	11.722	0.255	0.181	0.045	0.399	18.9
9-10	2.22	0.050	11.726	0.278	0.183	0.049	0.510	20.7
9-11	2.25	0.052	11.460	0.301	0.197	0.053	0.615	22.4
9-12	1.93	0.056	11.334	0.200	0.170	0.040	0.013	14.9
9-13	2.07	0.052	11.669	0.217	0.177	0.043	0.086	16.1
9-14	2.03	0.055	11.267	0.237	0.185	0.047	0.180	17.6
9-15	2.19	0.055	11.252	0.262	0.195	0.052	0.425	19.5
9-16	2.30	0.054	11.214	0.290	0.205	0.058	0.661	21.6
9-17	2.40	0.054	11.099	0.323	0.216	0.064	0.741	24.0
9-18	2.41	0.059	10.427	0.368	0.231	0.073	0.510	27.3

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

LIU & CARTER- 4.3MM, MIXTURE

TEST	VMEAN	F	C	RB	Y*B	Y	PARTI	RB/D50
1- 1	2.84	0.016	21.905	0.298	0.130	0.022	0.017	21.1
1- 2	3.03	0.019	19.678	0.328	0.154	0.031	0.011	23.2
1- 3	3.12	0.017	20.697	0.329	0.151	0.030	0.013	23.3
1- 4	2.72	0.016	22.404	0.241	0.121	0.019	0.015	17.1
1- 5	2.87	0.018	20.827	0.261	0.133	0.025	0.012	18.5
1- 6	2.88	0.017	21.017	0.272	0.137	0.025	0.013	19.3
1- 7	2.87	0.016	21.664	0.273	0.133	0.023	0.014	19.3
1- 8	3.01	0.016	22.093	0.275	0.136	0.024	0.014	19.5
1- 9	3.11	0.013	0.000	0.000	0.000	0.000	0.000	0.0
1-10	3.18	0.016	22.013	0.288	0.144	0.027	0.065	20.4
1-11	3.23	0.021	18.367	0.321	0.179	0.042	0.044	22.7
1-12	3.05	0.014	0.000	0.000	0.000	0.000	0.000	0.0
1-13	3.49	0.014	0.000	0.000	0.000	0.000	0.000	0.0
1-14	3.47	0.009	0.000	0.000	0.000	0.000	0.000	0.0
1-15	3.38	0.009	0.000	0.000	0.000	0.000	0.000	0.0
1-16	3.38	0.007	0.000	0.000	0.000	0.000	0.000	0.0
1-17	3.50	0.007	0.000	0.000	0.000	0.000	0.000	0.0
1-18	3.61	0.003	0.000	0.000	0.000	0.000	0.000	0.0
1-19	3.40	0.010	0.000	0.000	0.000	0.000	0.000	0.0
1-20	3.42	0.006	0.000	0.000	0.000	0.000	0.000	0.0
1-21	3.42	0.011	0.000	0.000	0.000	0.000	0.000	0.0
1-22	3.80	0.004	0.000	0.000	0.000	0.000	0.000	0.0
1-23	3.54	0.008	0.000	0.000	0.000	0.000	0.000	0.0
1-24	3.51	0.011	0.000	0.000	0.000	0.000	0.000	0.0
1-25	3.51	0.006	0.000	0.000	0.000	0.000	0.000	0.0
1-26	3.12	0.005	0.000	0.000	0.000	0.000	0.000	0.0
1-27	3.73	0.008	0.000	0.000	0.000	0.000	0.000	0.0
1-28	3.71	0.010	0.000	0.000	0.000	0.000	0.000	0.0
1-29	2.47	0.024	17.961	0.148	0.138	0.025	0.025	10.5
1-30	2.58	0.023	18.327	0.154	0.141	0.026	0.021	10.9
1-31	2.63	0.023	18.077	0.165	0.146	0.028	0.036	11.7
1-32	2.79	0.021	19.275	0.172	0.145	0.027	0.022	12.2
1-33	2.88	0.019	20.235	0.180	0.142	0.027	0.095	12.7
1-34	3.02	0.024	17.446	0.195	0.173	0.039	0.079	13.8
1-35	3.00	0.023	17.828	0.205	0.168	0.037	0.181	14.5
1-36	3.08	0.024	17.649	0.205	0.174	0.040	0.216	14.5
1-37	3.03	0.024	17.637	0.218	0.172	0.039	0.238	15.5
1-38	3.30	0.022	18.475	0.220	0.179	0.042	0.629	15.6
1-39	3.52	0.018	20.412	0.222	0.173	0.039	0.826	15.7
1-40	3.49	0.020	19.250	0.228	0.182	0.043	0.767	16.1
1-41	3.62	0.020	19.542	0.238	0.186	0.045	1.335	16.9
1-42	3.20	0.024	18.138	0.097	0.176	0.041	0.045	6.9
1-43	3.37	0.018	21.324	0.097	0.153	0.033	0.182	6.9
1-44	3.44	0.015	0.000	0.000	0.000	0.000	0.000	0.0
1-45	3.46	0.017	21.634	0.106	0.160	0.034	0.845	7.5
1-46	3.40	0.018	20.727	0.114	0.164	0.035	0.800	8.1
1-47	3.65	0.012	0.000	0.000	0.000	0.000	0.000	0.0
1-48	3.41	0.010	0.000	0.000	0.000	0.000	0.000	0.0

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

LIU & CARTER- 4.3MM, MIXTURE

TEST	VMEAN	F	C	RP	Y*B	Y	PART1	RB/D50
1-49	3.58	0.018	20.962	0.122	0.171	0.038	0.988	8.7
1-50	3.75	0.016	22.222	0.121	0.169	0.037	0.835	8.6
1-51	2.80	0.010	0.000	0.000	0.000	0.000	0.000	0.0
1-52	2.59	0.034	15.100	0.094	0.172	0.039	0.010	6.6
1-53	2.63	0.037	14.353	0.104	0.183	0.044	0.044	7.4
1-54	2.80	0.031	15.597	0.105	0.179	0.042	0.087	7.5
1-55	2.84	0.032	15.474	0.107	0.184	0.044	0.214	7.6
1-56	2.83	0.022	19.010	0.106	0.149	0.029	0.310	7.5
1-57	2.95	0.032	15.352	0.109	0.185	0.045	0.222	7.7
1-58	2.99	0.031	15.731	0.112	0.190	0.047	0.461	8.0
1-59	3.17	0.029	16.296	0.122	0.194	0.049	0.773	8.6
1-60	3.19	0.030	15.997	0.138	0.200	0.052	1.286	9.8
1-61	3.58	0.025	17.519	0.148	0.205	0.055	1.352	10.5

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

LIU & CARTER- 3.25M, MIXTURE

TEST	VMEAN	F	C	RB	Y*B	Y	PARTI	RB/D50
2- 1	2.32	0.012	0.000	0.000	0.000	0.000	0.000	0.0
2- 2	2.42	0.014	0.000	0.000	0.000	0.000	0.000	0.0
2- 3	2.47	0.014	0.000	0.000	0.000	0.000	0.000	0.0
2- 4	2.55	0.014	0.000	0.000	0.000	0.000	0.000	0.0
2- 5	2.72	0.010	0.000	0.000	0.000	0.000	0.000	0.0
2- 6	2.66	0.013	0.000	0.000	0.000	0.000	0.000	0.0
2- 7	2.66	0.017	21.470	0.272	0.124	0.027	0.191	25.5
2- 8	2.79	0.015	23.005	0.269	0.121	0.026	0.109	25.3
2- 9	2.85	0.014	23.504	0.269	0.121	0.025	0.150	25.2
2-10	2.97	0.016	21.783	0.288	0.136	0.032	0.187	27.0
2-11	2.96	0.018	19.987	0.304	0.148	0.038	0.262	28.5
2-12	3.08	0.017	20.674	0.306	0.149	0.038	0.299	28.7
2-13	3.14	0.017	21.069	0.306	0.149	0.039	0.295	28.7
2-14	2.32	0.021	19.917	0.187	0.123	0.026	0.026	17.5
2-15	2.36	0.017	21.392	0.188	0.110	0.021	0.026	17.7
2-16	2.52	0.017	21.243	0.198	0.118	0.024	0.047	18.6
2-17	2.60	0.017	21.618	0.204	0.120	0.025	0.084	19.2
2-18	2.66	0.015	0.000	0.000	0.000	0.000	0.000	0.0
2-19	2.75	0.017	21.629	0.220	0.128	0.028	0.292	20.7
2-20	2.82	0.013	0.000	0.000	0.000	0.000	0.000	0.0
2-21	2.85	0.016	22.564	0.225	0.126	0.028	0.624	21.1
2-22	2.89	0.015	22.656	0.230	0.128	0.028	0.670	21.6
2-23	2.86	0.018	20.500	0.242	0.139	0.034	0.628	22.7
2-24	3.00	0.016	22.203	0.237	0.135	0.032	0.775	22.2
2-25	3.14	0.015	22.841	0.244	0.137	0.033	1.032	22.9
2-26	3.16	0.013	0.000	0.000	0.000	0.000	0.000	0.0
2-27	3.25	0.012	0.000	0.000	0.000	0.000	0.000	0.0
2-28	3.30	0.015	22.506	0.267	0.147	0.037	1.907	25.1
2-29	3.56	0.013	0.000	0.000	0.000	0.000	0.000	0.0
2-30	2.51	0.025	17.359	0.130	0.144	0.036	0.042	12.2
2-31	2.28	0.029	16.271	0.153	0.140	0.034	0.013	14.3
2-32	2.41	0.028	16.430	0.147	0.146	0.037	0.012	13.8
2-33	2.68	0.024	17.084	0.138	0.149	0.039	0.167	13.0
2-34	2.69	0.025	17.589	0.145	0.153	0.041	0.273	13.6
2-35	2.43	0.026	16.948	0.150	0.145	0.037	0.018	14.9
2-36	2.72	0.025	17.631	0.148	0.154	0.041	0.333	13.9
2-37	2.72	0.025	17.365	0.159	0.157	0.043	0.516	14.9
2-38	2.80	0.024	17.636	0.163	0.159	0.044	0.613	15.3
2-39	2.89	0.024	17.972	0.165	0.161	0.045	0.672	15.5
2-40	2.99	0.023	18.050	0.172	0.165	0.047	0.944	16.1
2-41	3.09	0.022	18.496	0.176	0.167	0.049	1.174	16.5
2-42	3.21	0.021	19.178	0.179	0.167	0.048	1.414	16.8
2-43	3.35	0.020	19.750	0.187	0.170	0.050	1.484	17.5
2-44	3.43	0.019	20.149	0.196	0.170	0.050	1.851	18.4
2-45	2.11	0.040	13.947	0.077	0.152	0.040	0.025	7.2
2-46	2.11	0.043	13.334	0.083	0.158	0.043	0.024	7.7
2-47	2.27	0.036	14.669	0.082	0.155	0.042	0.159	7.7
2-48	2.22	0.038	14.161	0.084	0.157	0.043	0.232	7.9

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

LIU & CARTER- 3.25M, MIXTURE

TEST	VMEAN	F	C	RB	Y*R	Y	PART1	RB/D50
2-49	2.30	0.036	14.728	0.084	0.156	0.042	0.105	7.9
2-50	2.36	0.037	14.474	0.088	0.163	0.046	0.453	8.2
2-51	2.36	0.040	13.896	0.094	0.170	0.050	0.372	8.9
2-52	2.46	0.038	14.160	0.098	0.173	0.052	0.592	9.2
2-53	2.56	0.032	15.473	0.095	0.165	0.047	1.059	8.9
2-54	2.53	0.032	15.597	0.094	0.165	0.047	0.606	8.9
2-55	2.61	0.032	15.394	0.099	0.169	0.050	0.959	9.3
2-56	2.62	0.032	15.576	0.103	0.168	0.049	1.059	9.6
2-57	2.82	0.027	17.072	0.097	0.165	0.048	1.461	9.1
2-58	2.67	0.033	15.103	0.106	0.177	0.054	1.258	9.9
2-59	2.87	0.029	16.153	0.107	0.178	0.055	1.370	10.0
2-60	2.81	0.031	15.571	0.112	0.180	0.056	1.259	10.5

TABLE A - 5

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

LIU & CARTER- 2.26M, MIXTURE

TEST	VMEAN	F	C	RB	Y*B	Y	PART1	RB/D50
3- 1	2.03	0.019	20.492	0.185	0.099	0.024	0.015	24.9
3- 2	2.10	0.021	19.249	0.200	0.103	0.030	0.012	27.0
3- 3	2.19	0.018	20.937	0.205	0.104	0.027	0.046	27.7
3- 4	2.16	0.018	20.614	0.220	0.105	0.027	0.180	29.7
3- 5	2.35	0.020	19.259	0.232	0.122	0.037	0.128	31.3
3- 6	2.51	0.019	20.007	0.229	0.126	0.039	0.371	32.2
3- 7	2.50	0.022	18.514	0.252	0.135	0.045	0.307	34.0
3- 8	3.53	0.008	0.000	0.000	0.000	0.000	0.000	0.0
3- 9	1.38	0.090	7.750	0.567	0.179	0.080	0.349	76.4
3-10	2.55	0.019	20.525	0.273	0.124	0.038	0.723	36.9
3-11	2.77	0.017	21.431	0.267	0.127	0.042	1.147	35.9
3-12	2.93	0.015	22.834	0.267	0.126	0.040	1.687	36.0
3-13	3.02	0.017	21.172	0.281	0.143	0.051	1.687	37.9
3-14	2.99	0.014	23.874	0.271	0.125	0.039	2.987	36.6
3-15	3.02	0.018	20.674	0.294	0.146	0.053	2.512	39.7
3-16	2.01	0.025	17.446	0.158	0.115	0.033	0.015	21.3
3-17	2.25	0.021	19.394	0.167	0.116	0.033	0.018	22.5
3-18	2.26	0.021	19.051	0.175	0.119	0.035	0.044	23.6
3-19	2.27	0.021	19.376	0.180	0.120	0.036	0.244	24.3
3-20	2.34	0.021	19.202	0.184	0.122	0.037	0.188	24.8
3-21	2.39	0.020	19.366	0.190	0.124	0.038	0.318	25.6
3-22	2.40	0.021	18.826	0.202	0.128	0.041	0.556	27.3
3-23	2.49	0.021	19.230	0.209	0.130	0.042	0.837	28.2
3-24	2.56	0.020	19.685	0.210	0.130	0.042	1.117	28.3
3-25	2.59	0.021	18.879	0.217	0.137	0.047	0.942	29.3
3-26	2.69	0.019	20.350	0.218	0.132	0.044	1.537	29.3
3-27	2.72	0.021	19.112	0.222	0.143	0.051	1.363	29.9
3-28	2.69	0.020	19.488	0.228	0.138	0.048	1.758	30.8
3-29	2.86	0.018	20.805	0.225	0.137	0.047	1.785	30.4
3-30	2.88	0.019	19.797	0.226	0.145	0.053	2.631	30.5
3-31	2.92	0.016	22.010	0.220	0.133	0.044	3.562	29.6
3-32	2.06	0.024	18.260	0.080	0.113	0.032	0.020	10.8
3-33	2.14	0.025	17.633	0.091	0.121	0.037	0.080	12.3
3-34	2.14	0.026	17.314	0.095	0.124	0.038	0.115	12.8
3-35	2.22	0.025	17.805	0.096	0.125	0.039	0.415	13.0
3-36	2.21	0.023	18.651	0.101	0.118	0.035	0.293	13.6
3-37	2.34	0.023	18.513	0.100	0.127	0.040	0.769	13.4
3-38	2.43	0.020	19.855	0.099	0.123	0.037	1.187	13.4
3-39	2.31	0.024	17.850	0.109	0.129	0.042	0.846	14.6
3-40	2.27	0.027	17.011	0.111	0.134	0.044	1.105	15.0
3-41	2.36	0.026	17.166	0.118	0.138	0.047	1.647	15.9
3-42	2.45	0.023	18.438	0.113	0.133	0.044	2.063	15.3
3-43	2.53	0.023	18.324	0.122	0.138	0.047	1.934	16.4
3-44	2.55	0.022	18.612	0.122	0.138	0.047	2.598	16.5
3-45	2.60	0.024	19.128	0.128	0.144	0.051	2.579	17.3
3-46	2.73	0.023	18.407	0.137	0.148	0.055	3.264	18.5
3-47	1.87	0.039	14.120	0.054	0.133	0.044	0.058	7.3
3-48	1.97	0.037	14.566	0.057	0.135	0.046	0.299	7.7

TABLE A - 5

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

LIU & CARTER- 2.26M, MIXTURE

TEST	VMEAN	F	C	RB	V*P	Y	PART1	RB/D50
3-49	2.00	0.037	14.541	0.060	0.137	0.047	0.331	8.1
3-50	1.95	0.044	13.327	0.067	0.146	0.053	0.453	9.0
3-51	2.03	0.039	14.093	0.065	0.144	0.052	0.686	8.7
3-52	2.15	0.036	14.616	0.067	0.147	0.054	1.420	9.1
3-53	2.18	0.037	14.425	0.071	0.151	0.057	1.608	9.6
3-54	2.26	0.034	15.131	0.071	0.149	0.056	1.825	9.6
3-55	2.22	0.039	14.100	0.077	0.157	0.062	2.189	10.3

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

LIU & CARTER- 3.60M, UNIFORM

TEST	VMEAN	F	C	RB	Y*R	Y	PART1	RB/D50
5- 1	2.46	0.024	17.561	0.253	0.140	0.031	0.009	21.4
5- 2	2.54	0.026	16.608	0.279	0.153	0.036	0.015	23.6
5- 3	2.83	0.019	20.073	0.274	0.141	0.031	0.023	23.2
5- 4	2.80	0.016	21.693	0.280	0.129	0.026	0.046	23.7
5- 5	2.93	0.015	23.006	0.280	0.127	0.025	0.069	23.7
5- 6	2.35	0.022	18.542	0.195	0.127	0.025	0.008	15.7
5- 7	2.45	0.024	17.320	0.202	0.137	0.029	0.007	17.1
5- 8	2.62	0.025	17.231	0.217	0.153	0.037	0.012	18.4
5- 9	2.72	0.023	17.979	0.230	0.151	0.036	0.020	19.4
5-10	2.83	0.021	18.821	0.234	0.150	0.035	0.030	19.8
5-11	2.85	0.020	19.525	0.245	0.146	0.033	0.035	20.7
5-12	2.87	0.019	20.153	0.243	0.143	0.032	0.076	20.6
5-13	2.90	0.020	19.326	0.262	0.145	0.033	0.105	22.2
5-14	3.04	0.018	20.903	0.248	0.145	0.033	0.326	21.0
5-15	2.97	0.019	19.662	0.273	0.151	0.036	0.302	23.1
5-16	3.13	0.019	19.723	0.284	0.159	0.039	0.406	24.1
5-17	2.50	0.025	17.416	0.126	0.144	0.032	0.015	10.7
5-18	2.57	0.026	17.163	0.139	0.149	0.035	0.026	11.8
5-19	2.63	0.027	16.647	0.156	0.153	0.039	0.066	13.2
5-20	2.80	0.026	16.911	0.158	0.166	0.043	0.087	13.4
5-21	2.94	0.025	17.441	0.165	0.168	0.044	0.146	13.9
5-22	3.04	0.024	17.880	0.176	0.170	0.045	0.294	14.9
5-23	3.03	0.025	17.417	0.188	0.174	0.047	0.320	15.9
5-24	3.04	0.026	16.753	0.205	0.182	0.052	0.533	17.4
5-25	3.26	0.023	18.135	0.201	0.180	0.051	0.832	17.1
5-26	3.40	0.020	19.425	0.207	0.175	0.048	1.204	17.5
5-27	3.47	0.021	19.121	0.213	0.181	0.051	1.336	18.0
5-28	2.30	0.037	14.271	0.079	0.160	0.040	0.008	6.7
5-29	2.45	0.033	15.337	0.079	0.157	0.040	0.011	6.7
5-30	2.51	0.034	15.036	0.086	0.167	0.044	0.024	7.3
5-31	2.54	0.034	15.066	0.088	0.169	0.044	0.041	7.5
5-32	2.53	0.036	14.706	0.092	0.172	0.046	0.045	7.8
5-33	2.67	0.032	15.330	0.095	0.174	0.048	0.128	8.0
5-34	2.73	0.031	15.489	0.097	0.174	0.047	0.265	8.2
5-35	2.71	0.035	14.848	0.104	0.183	0.052	0.359	8.8
5-36	2.75	0.033	15.132	0.103	0.182	0.052	0.423	8.7
5-37	2.79	0.034	15.058	0.106	0.185	0.054	0.458	9.0
5-38	1.90	0.103	8.083	0.171	0.235	0.086	0.371	14.5
5-39	3.00	0.028	16.179	0.107	0.185	0.054	0.793	9.0
5-40	2.85	0.035	14.736	0.116	0.194	0.059	0.867	9.9
5-41	2.88	0.036	14.523	0.122	0.198	0.061	1.179	10.3

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

MEYER-PETER&BARYTA - 5.21MM, UNIFORM

TEST	VMEAN	F	C	RB	Y*B	Y	PART1	RB/D50
1- 1	3.64	0.049	11.748	0.163	0.310	0.054	0.160	9.5
1- 2	3.47	0.049	11.698	0.170	0.296	0.059	0.047	10.0
1- 3	3.79	0.054	11.131	0.159	0.340	0.065	0.386	9.3
1- 4	3.97	0.025	12.696	0.369	0.313	0.055	0.167	21.6
1- 5	4.81	0.043	11.590	0.325	0.416	0.098	1.181	19.0
1- 6	4.55	0.028	12.340	0.334	0.369	0.077	0.851	19.5
1- 7	3.84	0.031	12.958	0.477	0.298	0.050	0.051	27.9
1- 8	4.60	0.038	11.731	0.433	0.392	0.087	0.722	25.3

TABLE A - 6

ANALYSIS OF COARSE BED-MATERIAL FLUME DATA

MEYER-PETER&LIGNITE- 5,21MM, UNIFORM

TEST	VMEAN	F	C	RB	V*B	Y	PART1	RB/D50
1- 1	1,21	0,054	10,866	0,219	0,111	0 090	0 607	12,8
1- 2	1,46	0,060	10,455	0,187	0,140	0.142	1,950	10,9
1- 3	1,59	0,062	10,327	0,173	0,154	0.173	2,571	10 1
1- 4	1,40	0,084	9,203	0,103	0,152	0.168	2,709	6,0
1- 5	1,71	0,057	9,965	0,328	0,172	0,215	1,907	19,2
1- 6	1,01	0,066	10,294	0,139	0,098	0,070	0,218	8,1
1- 7	1,13	0,047	11,677	0,228	0,097	0,068	0,226	13,3
1- 8	1,82	0,064	10,268	0,153	0,177	0.229	3,745	9,0
1- 9	2,05	0,050	11,025	0,275	0,186	0,251	3,330	16,1
1-10	0,81	0,155	6,835	0,076	0,118	0,101	0,524	4,4
1-11	0,74	0,150	7,103	0,043	0,105	0,080	0,751	2,5
1-12	0,93	0,079	9,949	0.025	0,093	0,063	1,086	1,5

APPENDIX 5

COMPUTATION OF BED-LOAD TRANSPORT USING BLENCH'S REGIME EQUATIONS

NORTH SASKATCHEWAN RIVER

AT

DRAYTON VALLEY

Computation of Bed-Load Discharge
Using Blench's Règime Equations

The following slope equation was used:

$$S = \frac{k\ F_{bo}^{11/12}\ f^{111}(c)}{K\ b^{1/6}\ Q^{1/12}}$$

where: (for cross-section 89)

$k = 2.00$ (transverse bars)

$S = 0.0015$

$Q = 35{,}000$ cfs

$K = \frac{3.63g}{\nu\ 1/4} = 1950$

$b_w = 775$ ft.

$F_{bo} = 4.6$ from D_{50} (number) $= 0.09$ ft.
or D_{50} (weight) $= 0.16$ ft.
and FIGURE 7.3 (Blench 1969)

$$f^{111}(c) = \frac{0.0015 \times 1950 \times (775)^{1/6} \times (35{,}000)^{1/12}}{2.00 \times (4.6)^{11/12}} = 2.75$$

From FIGURE 7.2 (Blench 1969)

C = 17 parts per 100,000 by weight.

Therefore

$$q_s = \frac{1}{1600} \times \frac{CxQ}{b_w} = \underline{\underline{0.48}}\ \text{lb/ft.sec.}$$

APPENDIX 6

RESISTANCE TO FLOW DATA
IMMOBILE CHANNELS

TABLE A-7
RESISTANCE TO FLOW DATA
IMMOBILE CHANNELS

RIVER	LOCATION	REFERENCE & PLOTTING SYMBOL	Q (cfs)	V_m (ft/sec)	S	A (ft²)	b_w (ft)	d_e (ft)	$D_{\cdot\cdot}$ (ft)	$\frac{d_e}{k}$ $k=2/3D_{50}$	V_* (ft/sec)	$\frac{V_m}{V_*}$	MANNING n	$\frac{n}{d_e^{1/6}}$
Quesnal	Lawless Creek	Kellerhals (1963)	14,800	11.40	0.00653	1,300	200	6.50	0.710	13.7	1.170	9.8	0.037	0.027
			5,340	7.40	0.00516	710	165	4.30	0.710	9.1	0.845	8.8	0.038	0.030
Chilko	Henry's Crossing		5,600	8.10	0.00503	690	150	4.60	0.470	14.7	0.863	9.4	0.036	0.028
			3,630	6.60	0.00503	522	141	3.70	0.470	11.8	0.773	8.6	0.038	0.031
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
Flume Tests	Test 1	Kellerhals (1963)	1.00	2.18	0.0120	0.476	3.4	0.14	0.025	8.4	0.233	9.4	0.020	0.028
	2		1.00	2.10	0.0099	0.572	2.6	0.22	0.025	13.2	0.265	7.9	0.023	0.030
	3		1.00	2.27	0.0124	0.450	3.0	0.15	0.025	9.0	0.245	9.2	0.021	0.027
	4		2.25	2.57	0.0063	0.884	3.4	0.26	0.025	15.6	0.229	11.2	0.019	0.024
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
U.S. Rivers		Barnes (1967)												
Blackfoot R.	Ovando, Mont.		8,200	6.93	0.0023	1,185	194	6.10	0.51	17.9	0.673	10.3	0.031	0.023
West Fork Bitteroot	Conner, Mont.		3,880	7.69	0.0046	505	105	4.80	0.57	12.6	0.842	9.1	0.036	0.028
Grande Ronde	LaGrande, Oreg.		4,620	7.45	0.0052	653	114	5.72	0.31	27.7	0.980	7.6	0.043	0.032
South F.Clear-water	Grangeville, Id.		12,600	9.44	0.0061	1,334	152	8.77	0.82	16.0	1.312	7.2	0.051	0.035
Rock Creek Canal	Darby, Mont.		138	4.22	0.0210	33	25	1.32	0.69	2.9	0.944	4.5	0.060	0.057
Merced R.	Yosemite, Calif.		1,950	6.50	0.0130	302	71	4.26	0.83	7.7	1.336	4.9	0.065	0.051
Boundary Cr.	Porthill, Idaho		2,530	7.19	0.0183	354	84	4.22	0.69	9.1	1.576	4.6	0.073	0.058
Alberta Rivers		Water Res. Alberta												
Sheep R.	Hwy. 2A		626	2.95	0.0015	212	145	1.46	0.12	18.2	0.265	11.1	0.025	0.023
Prairie Cr.	Rocky Mt. House		137	1.60	0.0012	88	78	1.13	0.18	9.5	0.209	7.7	0.035	0.034
James R.	Sundre		340	3.08	0.0028	111	100	1.11	0.12	13.9	0.316	9.8	0.028	0.028
Little Red Deer	Water Valley		339	3.05	0.0075	113	58	1.95	0.18	16.3	0.686	4.5	0.066	0.059
Highwood R.	Aldersyde		516	1.50	0.0008	340	184	1.85	0.21	13.2	0.218	6.9	0.042	0.038

TABLE A-7
RESISTANCE TO FLOW DATA
IMMOBILE CHANNELS

RIVER	LOCATION	REFERENCE & PLOTTING SYMBOL	Q	V_m	S	A	b_w	d_*	D_{80}	$\frac{d_*}{k}$ $k=2/3D_{80}$	V_*	$\frac{V_m}{V_*}$	MANNING n	$\frac{n}{d_*^{1/6}}$
			(cfs)	(ft/sec)		(ft²)	(ft)	(ft)	(ft)		(ft/sec)			
San Luis Valley Canals	Sec. 1	Lane and Carlson (1953) X	944	4.80	0.00271	196	66	3.00	0.270	16.7	0.522	9.2	0.033	0.027
			1500	5.88	0.00280	255	73	3.50	0.270	19.5	0.561	10.5	0.031	0.025
	2		366	3.91	0.00375	94	52	1.80	0.253	10.0	0.466	8.4	0.036	0.033
			668	5.83	0.00376	115	55	2.09	0.253	12.4	0.503	11.6	0.027	0.024
	4		456	4.65	0.00362	98	47	2.08	0.250	12.5	0.492	9.5	0.032	0.029
			768	6.53	0.00359	117	48	2.44	0.250	14.6	0.530	12.3	0.025	0.022
	5		289	4.36	0.00371	66	37	1.78	0.177	15.1	0.461	9.5	0.030	0.027
			448	5.82	0.00368	77	40	1.92	0.177	16.3	0.476	12.2	0.026	0.023
	6		101	3.57	0.00299	28	21	1.29	0.138	14.0	0.353	10.1	0.027	0.026
			159	4.59	0.00295	35	22	1.60	0.138	17.4	0.390	11.8	0.022	0.020
	7		40.9	2.77	0.00290	15	15	1.00	0.135	11.1	0.305	9.1	0.029	0.029
			95.6	4.36	0.00290	22	16	1.38	0.135	15.4	0.358	12.2	0.022	0.021
	8		11.0	1.34	0.00319	8	18	0.46	0.128	5.4	0.217	6.2	0.036	0.041
			46.0	3.00	0.00316	15	19	0.80	0.128	9.4	0.285	10.5	0.024	0.025
	10		7.6	1.52	0.00973	5	11	0.47	0.210	3.4	0.383	4.0	0.059	0.067
			16.6	2.90	0.00965	6	11	0.52	0.210	3.7	0.402	7.2	0.031	0.034
	11		124	2.71	0.00213	46	32	1.44	0.158	13.7	0.314	8.6	0.030	0.028
			203	3.88	0.00235	52	32	1.62	0.158	15.4	0.350	11.1	0.025	0.023
	12		99	3.53	0.00240	28	21	1.30	0.112	17.4	0.317	11.1	0.024	0.023
			128	4.00	0.00243	32	22	1.46	0.112	19.6	0.338	11.8	0.023	0.022
	14		64.7	2.56	0.00126	25	21	1.23	0.066	28.0	0.223	11.5	0.024	0.023
			110	3.29	0.00136	33	21	1.56	0.066	35.4	0.261	12.6	0.022	0.020
	15		201	3.65	0.00194	55	35	1.57	0.164	14.4	0.313	11.7	0.025	0.023
			477	4.84	0.00199	99	39	2.51	0.164	23.0	0.400	12.1	0.026	0.022
	16		194	3.47	0.00202	56	34	1.66	0.161	15.5	0.328	10.6	0.024	0.022
	17		197	3.68	0.00267	54	37	1.45	0.125	17.4	0.350	10.5	0.026	0.024
			531	5.51	0.00274	96	41	2.35	0.125	28.2	0.454	12.2	0.024	0.021
	18		62.5	2.21	0.00083	28	20	1.42	0.069	30.8	0.196	11.3	0.021	0.020
			235	3.80	0.00080	62	25	2.48	0.069	54.0	0.253	15.0	0.018	0.016
...	...	...	...	...	...	...	...	...	...	...	...	...	...	...

APPENDIX 7

RESISTANCE TO FLOW

FLUME TESTS WITH ARTIFICIAL CEMENTED BED AND NATURAL SORTED BED

DESCRIPTION OF FLUME

The one-foot wide wooden tilting flume at the University of Manitoba Hydraulics Laboratory is shown in PHOTOGRAPH A-1. This flume is 37 feet long and is mounted on a truss, whose slope can be adjusted by a hand cranked shaft. Along the top of the flume are mounted steel angles which support a movable point-gauge carraige. The point-gauge can measure vertical elevations relative to the flume to the nearest 0.001 foot. A slotted end-gate is used to control the water surface profile in the flume. The water supply pipe at the inlet is fitted with a calibrated orifice plate for discharge measurement.

PREPARATION OF THE ARTIFICIAL CEMENTED BED

The material used in the flume tests consisted of gravel from 1/4 to 3/4 inches in size. Sizes smaller than 1/4 inches were excluded from the mixture.

The mixture was cemented to plexiglass sheets which were placed in a wooden form. The sheets were coated with a contact cement (3M Ten Bond) and a four inch layer of gravel was tamped into place over the sheets. After the cement hardened (usually 24 hours) the loose stones were removed and the plexiglass sheets were placed into the flume. In order to prevent movement of the sheets small holes were drilled through the sheets

and these were nailed to the wooden bottom of the flume.

PREPARATION OF THE NATURAL GRAVEL BED

The gravel mixture was placed in the flume to a depth of approximately 2 inches. This gravel layer was levelled and tamped into place. Water, having a fairly high velocity, was then introduced into the flume to allow the gravel to move into a natural position. Some gravel was removed from the flume by the flow, which took place for 2 hours, but at no location was the thickness of the remaining mixture less than 1 inch. The surface gravel particles would then be sorted by the flowing water, in fact, many of the particles were found to have their long axis perpendicular to the direction of flow as in actual coarse-bed rivers.

TABLE A-8

RESISTANCE TO FLOW-ARTIFICAL CEMENTED BED

Flume Width = 1.0 ft., $\nu = 1.41 \times 10^{-5}$ ft/sec.^{2}

Protrusion Height k = 0.024 ft.

TEST	SLOPE	Q ft^3/s	d ft.	R ft.	Vm ft/s	fb	V_m/v_{*b}	Rb ft.	R_b/k
A-1	0.00334	0.290	0.271	0.176	1.07	0.203	6.26	0.248	11.8
A-2	0.00334	0.190	0.232	0.158	0.82	0.279	5.35	0.218	10.4
A-3	0.00334	0.265	0.249	0.167	1.06	0.171	6.82	0.227	10.8
A-4	0.00334	0.152	0.203	0.144	0.75	0.296	5.17	0.194	9.2
A-5	0.00334	0.092	0.165	0.124	0.56	-	-	-	-
B-1	0.00244	0.260	0.285	0.181	0.91	0.216	6.05	0.258	12.3
B-2	0.00244	0.210	0.250	0.167	0.84	0.169	6.84	0.228	18.8
B-3	0.00244	0.152	0.229	0.148	0.66	0.286	5.26	0.222	9.6
B-4	0.00244	0.171	0.222	0.154	0.77	0.317	5.01	0.210	10.0
B-5	0.00244	0.176	0.233	0.159	0.76	0.239	5.76	0.217	10.4
B-6	0.00244	0.152	0.222	0.154	0.69	0.280	5.32	0.210	10.0
B-7	0.00244	0.052	0.148	0.114	0.35	-	-	-	-
C-1	0.00183	0.272	0.286	0.182	0.95	0.130	7.80	0.250	11.9
C-2	0.00183	0.240	0.280	0.180	0.86	0.160	7.07	0.250	11.9
C-3	0.00183	0.206	0.265	0.173	0.78	0.186	6.54	0.240	11.4
C-4	0.00183	0.162	0.241	0.163	0.67	0.230	5.87	0.222	10.6
C-5	0.00183	0.100	0.199	0.142	0.50	-	-	-	-
C-6	0.00183	0.058	0.149	0.115	0.39	-	-	-	-
D-1	0.00122	0.276	0.327	0.198	0.84	0.123	8.02	0.297	14.1
D-2	0.00122	0.220	0.343	0.203	0.64	0.236	5.78	0.308	14.7
D-3	0.00122	0.270	0.339	0.202	0.80	0.145	7.38	0.292	13.9
D-4	0.00122	0.225	0.315	0.193	0.71	0.171	6.83	0.276	13.2
D-5	0.00122	0.191	0.268	0.175	0.71	0.140	7.51	0.236	11.3
D-6	0.00122	0.151	0.265	0.173	0.57	0.234	5.81	0.241	11.5
D-7	0.00122	0.135	0.240	0.162	0.56	0.217	6.06	0.218	10.4
D-8	0.00122	0.058	0.191	0.138	0.30	-	-	-	-
E-1	0.00061	0.264	0.350	0.206	0.76	0.076	10.03	0.274	13.1
E-2	0.00061	0.234	0.356	0.208	0.66	0.105	8.62	0.238	11.4
E-3	0.00061	0.192	0.341	0.202	0.56	0.143	7.40	0.246	11.7
E-4	0.00061	0.160	0.306	0.192	0.47	0.140	7.51	0.248	11.8
E-5	0.00061	0.120	0.271	0.176	0.44	0.195	6.39	0.244	11.6
E-6	0.00061	0.062	0.219	0.153	0.28	-	-	-	-
E-7	0.00061	0.275	0.378	0.215	0.73	0.089	9.40	0.300	14.3

TABLE A-9

RESISTANCE TO FLOW-NATURAL SORTED BED

Flume Width = 1.0 ft., ν = 1.21 x 10^{-5} ft^2/sec.

Protrusion Height k = 0.030 ft.

TEST	SLOPE	Q ft^3/s	d ft.	R ft.	V_m ft/s	fb	V_m/V_{*b}	Rb ft.	R_b/k
F-1	0.00334	0.260	0.254	0.169	1.02	0.194	6.40	0.234	7.8
F-2	0.00334	0.245	0.250	0.167	0.98	0.207	6.25	0.230	7.7
F-3	0.00334	0.202	0.240	0.162	0.84	0.272	5.40	0.224	7.5
F-4	0.00334	0.158	0.208	0.147	0.76	0.293	5.20	0.196	6.5
F-5	0.00334	0.178	0.219	0.152	0.81	0.267	5.44	0.180	6.0
F-6	0.00334	0.108	0.183	0.134	0.59	-	-	-	-
G-1	0.00244	0.273	0.273	0.176	1.00	0.154	7.20	0.244	8.1
G-2	0.00244	0.264	0.269	0.175	0.95	0.170	6.81	0.245	8.2
G-3	0.00244	0.241	0.266	0.173	0.91	0.183	6.60	0.240	8.0
G-4	0.00244	0.282	0.254	0.169	0.80	0.234	5.83	0.236	7.9
G-5	0.00244	0.181	0.237	0.161	0.76	0.236	5.80	0.220	7.3
G-6	0.00244	0.159	0.223	0.154	0.71	0.255	5.57	0.208	6.9
G-7	0.00244	0.115	0.189	0.137	0.61	0.299	5.16	0.178	6.0
H-1	0.00183	0.272	0.293	0.185	0.93	0.139	7.51	0.266	8.9
H-2	0.00183	0.260	0.288	0.183	0.90	0.148	7.34	0.258	8.6
H-3	0.00183	0.233	0.283	0.181	0.82	0.176	6.73	0.256	8.6
H-4	0.00183	0.192	0.262	0.172	0.73	0.209	6.16	0.239	8.0
H-5	0.00183	0.152	0.233	0.158	0.65	0.236	5.80	0.215	7.2
H-6	0.00183	0.122	0.210	0.148	0.58	0.275	5.38	0.197	6.6
H-7	0.00183	0.073	0.193	0.139	0.38	-	-	-	-
I-1	0.00122	0.268	0.329	0.198	0.82	0.132	7.77	0.281	9.4
I-2	0.00122	0.251	0.318	0.194	0.79	0.138	7.56	0.276	9.2
I-3	0.00122	0.213	0.301	0.188	0.71	0.167	6.90	0.268	9.0
I-4	0.00122	0.178	0.260	0.171	0.69	0.154	7.17	0.231	7.7
I-5	0.00122	0.178	0.270	0.175	0.66	0.167	6.90	0.233	7.8
I-6	0.00122	0.140	0.253	0.168	0.55	0.237	5.80	0.195	6.5
I-7	0.00122	0.128	0.238	0.161	0.54	0.238	5.80	0.220	7.3
J-1	0.00061	0.278	0.369	0.212	0.76	0.079	10.00	0.287	9.6
J-2	0.00060	0.259	0.362	0.211	0.72	0.089	9.40	0.290	9.7
J-3	0.00061	0.214	0.337	0.201	0.64	0.108	8.57	0.280	9.3
J-4	0.00061	0.199	0.330	0.198	0.60	0.118	8.20	0.272	9.1
J-5	0.00061	0.178	0.324	0.197	0.50	0.145	7.40	0.272	9.1
J-6	0.00061	0.150	0.294	0.185	0.51	0.154	7.16	0.268	8.9
J-7	0.00061	0.110	0.240	0.162	0.46	0.158	7.07	0.213	7.1

TABLE A-10

RESISTANCE TO FLOW NATURAL SORTED BED
MANNING'S η COMPUTATION

Flume Width = 1.0 ft., $\nu = 1.21 \times 10^{-5}$ ft^2/s

Protrusion Height k = 0.030 ft.

TEST	Q ft^3/s	Rb ft.	Rb/k	η	$η/Rb^{1/6}$
F-1	0.260	0.234	7.8	0.032	0.041
F-2	0.245	0.230	7.7	0.033	0.043
F-3	0.202	0.224	7.5	0.037	0.048
F-4	0.158	0.196	6.5	0.037	0.050
F-5	0.178	0.180	6.0	0.046	0.062
F-6	0.108	-	-	-	-
G-1	0.273	0.244	8.1	0.029	0.036
G-2	0.264	0.245	8.2	0.030	0.038
G-3	0.241	0.240	8.0	0.031	0.040
G-4	0.202	0.236	7.9	0.035	0.045
G-5	0.181	0.220	7.3	0.035	0.045
G-6	0.159	0.208	6.9	0.036	0.047
G-7	0.115	0.178	6.0	0.039	0.051
H-1	0.272	0.266	8.9	0.029	0.035
H-2	0.260	0.258	8.6	0.029	0.036
H-3	0.233	0.256	8.6	0.031	0.039
H-4	0.192	0.239	8.6	0.033	0.042
H-5	0.152	0.215	7.2	0.035	0.045
H-6	0.122	0.197	6.6	0.038	0.049
H-7	0.073	-	-	-	-
I-1	0.268	0.281	9.4	0.028	0.034
I-2	0.251	0.276	9.2	0.028	0.034
I-3	0.213	0.268	9.0	0.031	0.037
I-4	0.178	0.231	7.7	0.029	0.036
I-5	0.178	0.233	7.8	0.030	0.038
I-6	0.140	0.195	6.5	0.032	0.042
I-7	0.128	0.220	7.3	0.035	0.045
J-1	0.278	0.287	9.6	0.021	0.025
J-2	0.259	0.290	9.7	0.022	0.027
J-3	0.214	0.280	9.3	0.024	0.030
J-4	0.199	0.272	9.1	0.025	0.032
J-5	0.178	0.272	9.1	0.028	0.034
J-6	0.150	0.268	8.9	0.030	0.037
J-7	0.110	0.213	7.1	0.029	0.037

PHOTOGPAPH A-1

FLUME USED FOR RESISTANCE TO
FLOW STUDIES

APPENDIX 8

BANKFULL STAGE DATA
COARSE-BED CHANNELS

TABLE A-11

BANKFULL STAGE DATA

COARSE-BED CHANNELS

River	Q cfs	V_m ft/sec	S $\times 10^{-3}$	d ft.	W_s ft.	d_* ft.	D_{50} ft.	D_{90} ft.
Kellerhals (1963)								
Chilko R. at Henry's Crs.	5600	8.1	5.03	5.5	150	4.6	.47	.83
Taseko R. below Taseko Lake	6400	8.0	2.9	6.2	150	5.3	.49	1.0
Chilko R. at outlet of Chilko Lake	5000	3.4	.92		270	5.5	.37	.58
Cariboo R. at Quesnel Forks	12000	9.6	4.19	8.5	190	6.6	.88	1.5
Quesnel R. at Lawless Creek	14800	11.4	6.33	8.0	200	6.5	.71	1.3
Cariboo R. at outlet of Cariboo Lake	8000	4.7	1.92		260	6.6	.64	1.1
Galay (1967a)								
North Sask. River	80000	7.0	1.50		800	14.3	.10	
Alta.W.Res. (1967)								
Highwood R.	6000	5.5	1.55		205	5.3	.18	
Prairie R.	2900	6.5	3.20		100	4.5	.15	
James R.	7000	2.0	4.00		105	3.3	.12	
Sheep R.	6000	6.3	3.80		210	4.5	.11	

TABLE A-11

BANKFULL STAGE DATA

COARSE-BED CHANNELS

River	Location	Q cfs	V_m ft/sec	S $\times 10^{-3}$	d ft.	W_s ft.	d_x ft.	D_{50} ft.	D_{90} ft.
San	1	1500	5.88	2.80	4.87	73	3.50	.32	.70
Luis	2	668	5.83	3.76	2.81	55	2.08	.26	.50
Valley	4	768	6.53	3.59	3.11	48	2.54	.26	.53
Canals	5	448	5.82	3.68	2.50	40	1.92	.18	.28
(Lane	6	159	4.59	2.95	1.88	21.7	1.60	.14	.20
& Carlson	7	95.6	4.36	2.90	1.73	15.9	1.38	.14	.18
1953)	8	46	3.00	3.16	.96	19.2	.80	.13	.26
	9	-	-	-	-	-	-	.13	.19
x	10	16.6	2.90	9.65	.60	11.1	.515	.21	.40
	11	203	3.88	2.35	1.88	32.3	1.62	.17	.24
	12	128	4.00	2.43	1.77	21.9	1.46	.12	.17
	14	110	3.29	1.36	2.00	21.4	1.56	.07	.12
	15	477	4.84	1.99	3.05	39.4	2.50	.17	.28
	16	-	-	-	-	-	-	.17	.32
	17	531	5.51	2.74	2.60	41	2.35	.13	.17
	18	235	3.80	.80	2.94	25	2.48	.073	.10

APPENDIX 9

DEPTH OF SCOUR COMPUTATIONS
PRAIRIE CREEK

DEPTH OF SCOUR COMPUTATIONS

Prairie Creek Near Rocky Mountain House
(x-sec. 7+60 and 8+60 U/S)

Properties of Flow and Channel

Q_b = 2900 cfs

bw = 110 ft.

r = 240 ft.

θ = 90°

d_s = 9 ft.

D = 2.3 in. (grid sample by weight)

Lacey Equations (1929)

$d_* = 0.47\ (Q_b/_f)^{1/3}$

$f = \sqrt{64D} = 12.2$ (Questionable whether this can be applied to coarse-bed rivers).

$d_* = 0.47\ (\frac{2900}{12.2})^{1/3} = 2.9$ ft.

d_s = 2.9 x factor = 2.9 x 2 = 6 ft.

Blench Equations (see p. 122, 1969)

$d_{fo} = \frac{qf^{2/3}}{F_{bo}{}^{1/3}}$, $F_{bo} = 5.0$

$d_{fo} = (\frac{2900/110}{(5)\ 1/3})^{2/3} = 5.1$ ft.

ds = 5.1 x factor = 5.1 x 2 = 10 ft.

FIGURE 112 (free bend)

$r/_{bw\theta} = \frac{240}{110 \times 1.57} = 1.4$

From Figure 112, $ds/_{b_w}$ = 0.10

Therefore:

ds = 110 x 0.10 = 11 ft.

REQUEST FOR DUPLICATION

I wish a photocopy of the thesis by

F. 236

September 1975